SMALL STORM URBAN FLOW AND PARTICULATE WASHOFF CONTRIBUTIONS TO OUTFALL DISCHARGES

by

ROBERT ERVIN PITT

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Robert Ervin Pitt

Under the supervision of Professor William C. Boyle

Many governmental agencies are evaluating urban runoff problems. Urban runoff models play an important role in these evaluations. Unfortunately, many commonly used models incorrectly estimate runoff flows and the washoff of particulates from impervious surfaces during small rains. This research investigated these two processes in detail in two urban watersheds in Toronto, Ontario.

Runoff volume is the most important hydraulic parameter needed for water quality studies. Estimates of runoff volume were only found to require rain depth information. Both initial runoff abstractions (usually less than 1 mm) and continuous runoff losses (about 25 to 50 percent of the rain depth) were found to be important for impervious surfaces.

The general model for impervious area runoff developed during this research was shown to be applicable for a large variety of impervious surfaces and rain characteristics. This model was shown to be

related to both the SCS Curve Number procedure and the Horton infiltration equation, but it produced much smaller errors in runoff volume predictions due to variations in model parameter values.

This research also found that residue loading, rain depth, and rain intensity all significantly affected residue washoff from impervious surfaces. Only about 10 to 20 percent of the particulate residue on a surface could be removed by normal rains. Typical washoff prediction procedures used in urban runoff models greatly over-predict particulate residue washoff from impervious surfaces, especially for large particles. Because of this, particulate contributions from other source areas must be under-predicted to enable model calibration using monitored outfall data.

These research results were incorporated into a model that was applied to many different land uses and control practices. It was found that for small rains, impervious areas contributed most of the runoff flows and pollutants. However, specific development characteristics in a watershed (such as grass swales and roof connections) have dramatic affects on runoff volumes and flow rates.

Approved by William C. Boyle

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INTRODUCTION

A. Summary of Urban Runoff Water Quality Research

Urban runoff quality has been studied for at least 25 years in many countries, as discussed in Appendix A. By 1960, only a few studies concerning urban runoff quality had been conducted. During the 1970s, interest in urban runoff significantly increased due to the considerable expense associated with controlling municipal and industrial point sources. Legislators wanted to be certain that the large costs would result in improved receiving water quality, and that uncontrolled "nonpoint" discharges (specifically urban and agricultural runoff) would not sustain poor water quality conditions after the conventional point sources were controlled.

The US Environmental Protection Agency (EPA) funded many stormwater quality projects during the 1970s and early 1980s through its Storm and Combined

Sewer Section and the Water Planning Division. Much urban runoff quality data was also collected from 1978 through 1983 as part of the USEPA's Nationwide Urban Runoff Program (NURP). The Ontario Ministry of the Environment and Environment Canada have also sponsored important studies investigating urban runoff.

The brief history of urban runoff investigations presented in Appendix A is intended to present the shifts in emphasis of North American urban runoff research that has occurred during the past 15 years. An appropriate research approach for urban runoff would first document specific urban runoff problems by directly monitoring receiving water beneficial uses. Instead, early urban runoff research was intended to produce discharge yield estimates that could be compared to discharges from conventional point sources. The urban runoff yields observed were so large that it became politic to place emphasis on their control. It was not until the late 1970s that research was finally directed towards monitoring receiving water effects to document the actual needs for controlling urban runoff.

The second phase of an urban runoff research approach should be to document the specific sources of identified problem pollutants. This element has received little previous attention and is believed to be the weakest element of many urban runoff analyses.

The final phase of an urban runoff research approach should identify and evaluate the effectiveness of available control measures that can operate at the source areas (or at discharge locations) and remove the specific pollutants of concern. Much information on controls is available; if removal goals and source area yields are known, then appropriate control programs can be adequately designed.

B. Need to Simplify Urban Runoff Models

The history of urban runoff research has developed to an uncomfortable stage where many aspects of the processes involved in urban runoff have been investigated to some degree, but few comprehensive and critical reviews of this collective information have been completed. This has resulted in a proliferation of reports and papers that are individually interesting and informative, but taken together are not well enough coordinated to give a complete picture of all the processes involved in urban runoff.

Many of the model developers have not been involved in the field research that they rely upon. Thus urban runoff model developers often lack the insight needed to critically select the available information for inclusion in their models. Many of the urban runoff researchers have also not taken the effort to publish their work in reviewed journals. They commonly only publish their work in research reports which are not as readily available, or accepted. A large number of these research efforts have investigated detailed urban runoff processes, but in many cases the results are not easily transferable to other locations.

It appears that many urban runoff models contain superfluous elements that were included because of the model developers' attempts to be complete. It is often assumed that complex models are more accurate than simple models. This is not always true. Common problems with complex models include:

- o misuse for the problem at hand,
- o inappropriate process descriptions, and
- o extensive use of default parameter values which may be totally incorrect for the case under

study.

High costs involved in collecting calibration and model use input information, or the high costs required in operating the computer, may also make the use of complex models inappropriate. Sometimes it is difficult to accept the fact that a simpler model may be more suitable for the need at hand, and possibly more accurate.

Model developers and users must have well defined objectives for their modeling needs. In many cases, planners need to make stormwater management decisions without having much technical expertise or knowledge of the research information that was used in developing the model. The users must therefore rely on the model documentation which can be very complex. In many cases, adequate planning and even most design decisions can be made with models that are less precise and less complex than more comprehensive models.

McCuen (1986) summarized the need for simpler models during a keynote address at the 1986 Maryland Sediment and Stormwater Conference. He found that simple urban hydrology models containing few independent parameters can usually describe more than

80 percent of the variation of the predicted dependent parameters. He also found that complex models may be subject to larger errors due to the need to estimate many parameter values. He concluded that for many applications, simple models can provide results as good as, or better than, complex models.

A major objective of this dissertation research was to examine two major components of urban runoff modeling that have commonly received incorrect and overly complex treatment in many urban runoff models. The components investigated were the generation of flows during common small rains, and the washoff of particulates from impervious surfaces. This dissertation summarizes much of the previous research and the field testing and analyses conducted as part of this research that investigated these two major modeling components.

Available urban runoff models are not critiqued in this dissertation, but there is a critical evaluation of these two major modeling components mentioned above, as well as a comparison of the observed test results with the procedures that are commonly used in current models.

This information has been incorporated into a simplified model developed for stormwater management planners. Special versions of this model (the Source Loading and Management Model, or SLAMM) are currently being used as parts of the Toronto Area Wastewater Management Strategy Study conducted by the Ontario Ministry of the Environment and by the Wisconsin Department of Natural Resources in its priority watershed program.

C. Field Experience Conflicts with Modeling Concepts

Modeled Source Area Control Program Effectiveness May Not be Supported by Monitoring Results

Calibrated and verified urban runoff models have been used to estimate discharge yields in locations inadequately monitored (either for present or future conditions). This extrapolation procedure is commonly needed when examining large watersheds made up of many different land use areas and for many different control scenarios. Relatively large errors in drainage yields may occur, however, and significant decisions should be

based on actual field monitoring in the identified critical drainage areas.

Another important use of urban runoff models is in the design of control programs. The use of the models during the "208" studies (Areawide Wastewater Management Plans as required by Section 208 of the 1972 Water Pollution Control Act) resulted in very high expectations for runoff improvements from source area controls (especially street cleaning). The full scale street cleaning demonstration projects and the NURP studies, however, have shown much less runoff improvement from street cleaning (Pitt 1979; Pitt and Shawley 1982; Terstriep et al. 1982; Bannerman et al. 1983; EPA 1983; and Pitt 1984). The NURP studies did show substantial runoff improvements from well-designed detention basins that were predictable by the urban runoff models.

There are several potential reasons for these inconsistencies between field data and modeling results. The washoff of particulates from impervious areas is usually over-estimated by urban runoff models. Discharges of outfall solids are then balanced by under-estimating the importance of erosion losses from urban pervious areas. This results in the misrepresentation of source area yields and controls. As an example, street cleaning is usually much less effective than estimated by the models because of actual limited washoff during rains of the larger particulates that are preferentially removed by street cleaners (Bannerman et al. 1983; Pitt 1984). Data for source area runoff flows also tend to be incorrect, especially for impervious areas during small rains. The runoff losses for impervious areas are actually much greater than predicted by the models. The outfall runoff yields are then balanced by under-estimating the runoff from pervious areas. This again results in too much importance being given to impervious area yields.

Lack of Transferability of Modeling Processes Between "Flood" Hydrology Models and Water Quality Planning Models

An early paper by McPherson and Schneider (1974) warned of the common error of assuming that methods, techniques, and tools developed for flood and drainage analyses can be used for urban water quality planning analyses.

Stormwater management for many is restricted to the control of flooding and drainage problems. Water

quality and other environmental issues are usually not considered when developing a stormwater management system for a community. It is therefore not surprising that most urban runoff models are heavily based on flood and drainage analysis procedures. Over the years, water quality elements have been added to many of the early models. However, most newly developed urban runoff water quality models still use many of the assumptions and procedures that have been extensively used in analyzing large flows.

Urban runoff water quality and other environmental problems are associated more with the discharge of pollutants than with large flows (Pitt and Bozeman 1982). Two notable exceptions are the destruction of aquatic organism habitats and the flushing of polluted sediments by large receiving water flows (Pitt and Bissonnette 1984). Most of the pollutant discharges associated with urban runoff occur during common small rains (Pitt and McLean 1986). Rare, very large, rains can discharge massive pollutant quantities, but they occur infrequently, leading to small averaged annual loading contributions. The large rains that are important, from a pollutant discharge or flushing viewpoint, occur much more often (every several months) than the rare drainage or flood control design events (generally from 5 to 100 year events).

Care must be taken in using a model that stresses large events when conducting water quality analyses. These large event models often oversimplify runoff generation processes associated with small events. As examples, the initial runoff losses are often assumed to be a constant value, and the value is assumed to be quite small for urban areas with large amounts of impervious surfaces. These assumptions have little effect when predicting the runoff volumes for large rains, but it can dramatically affect the runoff predictions for small events. Similar problems occur when using the various infiltration models to predict runoff losses during a rain.

D. Research Needs

Water Balances in Urban Areas Need to be Better Understood

Davies and Hollis (1981) regretted the fact that so few urban runoff studies have examined water balances in urban areas, even after widespread

recognition of their value. They felt strongly that water balances are critical components of water cycle analysis in urban areas, especially if water quality is of concern. The design or analysis of source area urban runoff controls (for both quantity and quality) requires an understanding of the sources of the flows or pollutants of concern. A knowledge of source area pollutant contributions must start with an understanding of the source area runoff flow contributions. It is not possible to determine the effectiveness of treating parking lot runoff at a shopping mall, as an example, if the relative importance of the parking lot runoff in relation to other sources is not known.

Impervious Area Runoff Contributions

Falk and Niemczynowicz (1978) identified impervious surfaces as the most important contributors of flows in an urban area. They also stated that the hydrologic response of impervious surfaces is largely independent of geologic and climatic factors, possibly allowing good transferability of observations of impervious area runoff to a wide variety of situations. However, Ring (1983) believed that predicting street runoff (typically the most important impervious surface in urban areas) is the weakest link in designing a storm drainage system.

During common small rains, impervious surfaces contribute much of the flows and pollutants to an outfall. Because these small rains are also responsible for most of the flow and pollutant discharges, impervious surfaces acquire a great deal of importance in stormwater quality management.

Pollutant Washoff Mechanisms

Delleur (1983), in a summary of papers of urban runoff modeling processes, found a consensus among several authors of the need to improve the washoff prediction methods contained in popular urban runoff models. The early tests that examined particulate washoff from impervious surfaces have often been misused. It is generally assumed that the washoff models refer to the total particulate loadings on the impervious surface (total load), whereas they are usually only related to the total amount of particulates that can be washed off (available load). There can be a tenfold difference between total available load and total load.

This common error is further compounded by assuming very large particulate accumulation rates on impervious surfaces, a mistake caused by supposing zero initial loadings of street dirt after major rains or street cleaning. In all cases, relatively large initial loadings occur on impervious surfaces that cannot be removed by rains or street cleaning, but are removed by sampling procedures (Pitt 1979). The initial loadings are directly related to pavement texture; rough pavements have much greater initial loading values than smooth pavements (Pitt 1979; Pitt and Shawley 1982; Pitt 1984; Pitt and McLean 1986). Samples obtained several days after rains or street cleaning have been used to calculate very large initial particulate accumulation rates by assuming zero initial loading values.

The effect of these interpretation errors on a mass balance of pollutants in urban areas is a gross over-estimation of the importance of "removable" pollutants from impervious areas. Pollutant over-estimation is often linked to an over-estimation of runoff volumes from impervious areas (because of the usual assumption of no runoff losses from impervious surfaces). It is not surprising, then, that the "208" planning reports investigating urban runoff prepared in the 1970s grossly over-estimated the importance of street surface runoff and the effects of street cleaning on urban runoff quality.

E. Development of Hypotheses

Predictions of yields of urban runoff pollutants and their control usually rely on the use of urban runoff models. A good model requires an accurate representation of the sources of runoff flows and pollutants in the watershed. The hypothesis of this research is that current urban runoff models do not correctly predict the relative source flows and pollutant yields because of improper assumptions regarding small-storm urban hydrology and the washoff of particulates from impervious areas.

Many government agencies are currently evaluating local, regional, and national urban runoff problems. Inaccurate evaluations may result in inappropriate expenditures of large amounts of money, or ignorance of real problem areas. A better understanding of the sources and movements of urban runoff pollutants is an

important key in evaluating urban runoff problems and controls. It is hoped that this research may clarify some of the existing misunderstandings.

F. Organization of Dissertation

This dissertation research examined the relative contributions of flows and many pollutants from different source areas under a variety of weather and site conditions. Many small-scale observations of runoff volume and quality at source areas were supplemented with controlled washoff experiments and large-scale outfall monitoring. The data were used to identify the important variables and relationships affecting source area flow and pollutant contributions. These relationships were incorporated into SLAMM to enable the evaluation of the importance of different source areas in contributing flows and pollutants to the receiving waters and the effectiveness of different stormwater management practices.

Section 3 contains a discussion of the importance of knowing urban runoff flow and pollutant sources, and a description of the source oriented mass balance modeling concept used in SLAMM.

Sections 4 through 8 develop the urban hydrology source area models. Section 4 contains a series of exploratory data analyses that describe the basic structure of urban hydrology. Section 5 describes the special small-scale runoff tests conducted to identify significant environmental variables affecting runoff losses from impervious areas. Section 6 presents similar runoff loss models for impervious areas, but by using monitoring data from large shopping center parking areas and roofs, collected during a large variety of rains. Section 7 develops and calibrates a general paved area hydrology model for different impervious surfaces, using initial and variable runoff loss mechanisms identified in the earlier sections. Section 7 also compares this general paved area runoff model to the Soil Conservation Service curve number procedure and the Horton infiltration equation. Section 8 finally verifies the general impervious area model using independent outfall hydrology data collected from a variety of complex urban watersheds. Section 8 also develops simple hydrology relationships for pervious areas and demonstrates the use of the complete model to

estimate source contributions of flows and to calculate curve numbers for different development conditions and rains.

Section 9 describes the street dirt washoff tests that were conducted during this research and the general washoff model that was developed. This model is compared to other washoff models that have been used.

Section 10 summarizes the quality components of SLAMM, based on the extensive source area monitoring that was conducted during this research. This section also demonstrates how the completed model can be used to predict sources of pollutants in urban areas and to evaluate the effectiveness of urban runoff control programs.

SECTION 2
CONCLUSIONS

Several important conclusions involving urban runoff processes were documented during this dissertation research conducted in two urban watersheds in Toronto, Ontario. The most important research contributions concerned impervious area runoff during small rains and particulate washoff from impervious surfaces. Both of these processes are critical components of urban runoff quality models. Previous erroneous descriptions of these components have led model users to incorrect conclusions. This research was conducted to make the process descriptions used in modeling more reliable. The hypothesized processes were investigated on several scales and at many locations in the Toronto area. Added Milwaukee area data were used to confirm transferability of the processes according to different scales, land uses, rain depths, and geographical locations. The verified processes were then used in a complete urban runoff model to

demonstrate how different land development practices affect the sources of urban runoff pollutants and the selection of control programs. The following paragraphs briefly summarize these conclusions.

A. Simplified Approach for Urban Hydrology Modeling

1. Runoff volume is the most important hydraulic parameter needed for water quality studies, while water velocity (or stage) is the most important parameter for flooding and drainage studies. Common small rains account for much more of the annual runoff volume than rare flooding events.

2. Estimates of runoff volume were only found to require rain depth information. Other rain characteristics (including antecedent conditions, durations, intensities, etc.) did not substantially improve runoff volume predictions.

3. Both initial runoff abstractions (mostly detention/storage) and continuous runoff losses (infiltration) were found to be important for impervious surfaces. Impervious surface detention/storage values were constant for each surface

studied (they did not vary for different rain characteristics), and were generally less than 1 mm. Infiltration losses did vary substantially for different impervious surface and rain conditions and were about 25 to 50 percent of the rain depth.

4. The general model for impervious area runoff developed during this research was shown to be applicable for a large variety of impervious surfaces and rain characteristics.

5. The general model was shown to be related to both the SCS Curve Number procedure and the Horton infiltration equation.

6. The Horton equation, when applied to impervious surfaces, was shown to be related to rain intensity and not rain duration.

7. The selection of curve numbers and initial abstractions, even within the narrow range of accepted values for impervious surfaces (such as curve numbers of 95 to 98, and initial abstractions of 1 to 1.6 mm), in existing runoff models greatly affects predicted runoff for the small storms of most concern during water quality studies.

B. Residue Washoff Observations

1. Residue loading, rain depth, and rain intensity were all found to significantly affect residue washoff from impervious surfaces.

2. Almost all of the filterable residue was available for washoff from impervious surfaces, while only about ten percent of the particulate residue was ever washed from impervious surfaces.

3. Typical washoff prediction procedures used in urban runoff models greatly over-predict particulate residue washoff from impervious surfaces, especially for large particles.

4. If particulate washoff from impervious surfaces is over-predicted, then particulate contributions from other source areas must be under-predicted to enable model calibration using monitored outfall data.

5. Rains have a great preference for washing off small particles from impervious surfaces, as compared to large particles.

6. Small particles have much greater associated pollutant concentrations than large particles, but are not as common on impervious surfaces.

7. Many urban runoff control practices (such as catch basins, street cleaning, and wet detention basins) preferentially remove the more common, but not as polluted large particles.

C. Use of the Hydrology and Washoff Relationships in the Source Loading and Management Model

The hydrology and washoff relationships described above were used in the Source Loading and Management Model (SLAMM) to illustrate how this information can be used to help evaluate the importance of different source areas and the effectiveness of source area controls for different land development characteristics.

1. Directly connected impervious areas were found to contribute most of the runoff flows and pollutants during small rains which are of most concern for water quality studies. However, pervious areas contributed substantial flows and pollutants after about 10 to 25 mm of rain.

2. Specific development characteristics of a watershed were shown to have dramatic affects on runoff volume and flow rates, specifically:

- o the use of grass swales,
- o whether or not the roof drains or parking and storage areas were directly connected to the storm sewerage system,
- o the presence of alleys, and
- o the areas of the land cover elements.

3. A retro-fitting program, using a combination of infiltration and sedimentation practices, was identified as being cost-effective in the Humber River watershed. The program included the following elements:

- o wet detention basins serving 25 percent of the drainage area,
- o infiltration of runoff from half of the residential roofs currently draining to pavement, and
- o infiltration of runoff from half of the paved parking areas in high rise residential, non-manufacturing industrial, and commercial areas.

This program was estimated to cost about $400 per hectare per year and could achieve the following benefits:

pollutants	control (%)
bacteria	5 to 10%
flow, total residue, and filterable residue	15 to 20%
particulate residue, nutrients, COD, and heavy metals	30 to 45%

SECTION 3
KEY CONCEPTS

A. Importance of Urban Runoff Flow and Pollutant Sources

Urban runoff is comprised of many separate components that are combined at various locations above the discharge site before entering the receiving water. It may be adequate to consider the combined outfall conditions when evaluating the long-term, area-wide effects of many separate outfall discharges on a receiving water. However, if better predictions of outfall characteristics, or if source area control effectiveness predictions are needed, then the separate components must be recognized in a modeling effort.

Figure 3.1 is a schematic diagram showing the many component sources for a residential and light industrial area. This diagram shows three major sets of components; impervious areas, pervious areas, and the drainage system. The drainage system captures sheetflows from many sources, beginning at the roof

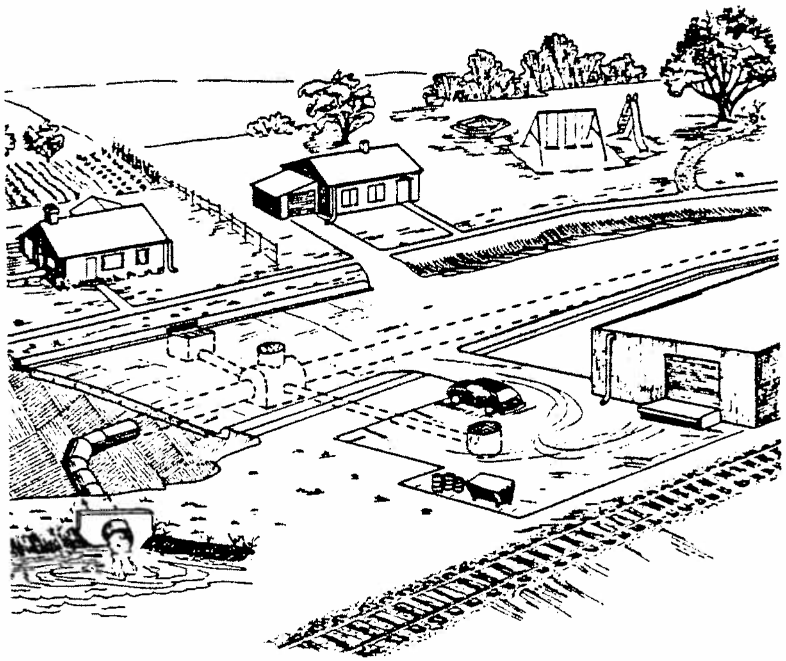


Figure 3.1 Urban Runoff Source Areas and Drainage System

gutters and downspouts. If these are discharged onto a paved area that in turn drains to road gutters and storm drain inlets, they are considered "directly connected" to the storm drain system. Some roof drains are connected to the household sanitary sewer connectors and would therefore not be a part of the storm drainage system. This practice is currently discouraged and many cities are actively disconnecting roof drains from the sanitary system. If the roof drains are discharged to pervious areas, much of their runoff flow could infiltrate and not contribute to the outfall discharges.

There are also several types of roadside drainage systems; paved or concrete curbs and gutters, sealed (paved) ditches, and grass swale ditches. Overland flow and street runoff enter these roadside drainages which direct the flows to storm drain inlets, or to open channels which flow to the receiving water. Some inlets may include catchbasin sumps that have more sediment accumulation potential than simple inlets. Inlets are often located in large paved areas (such as parking areas). Man-holes are usually located at street intersections where several connectors from close inlets are combined and the flows drop to the storm sewerage. Runoff is then discharged to the receiving water through an outfall. The outfall may be elevated above the receiving water, or submerged. If submerged, backwater effects can extend great distances up the sewerage.

The various source areas all contribute different amounts of flows and pollutants, depending on their specific characteristics. Impervious areas (such as paved parking lots, streets, driveways, roofs, and sidewalks) may contribute most of the runoff materials during small rains. Pervious areas (such as gardens, lawns, bare ground, unpaved parking areas and driveways, and undeveloped areas) become important contributors for larger rains. The relative importance of the sources is a function of their areas, their pollutant washoff potentials, and the rain characteristics. The outfall discharge is therefore made up of different source area contribution mixtures, depending on drainage area and rain characteristics. Source area control effectiveness is therefore highly site and rain specific.

Pollutant depositions and removals for the different source areas are shown on Figure 3.2. All areas are affected by atmospheric deposition sources,

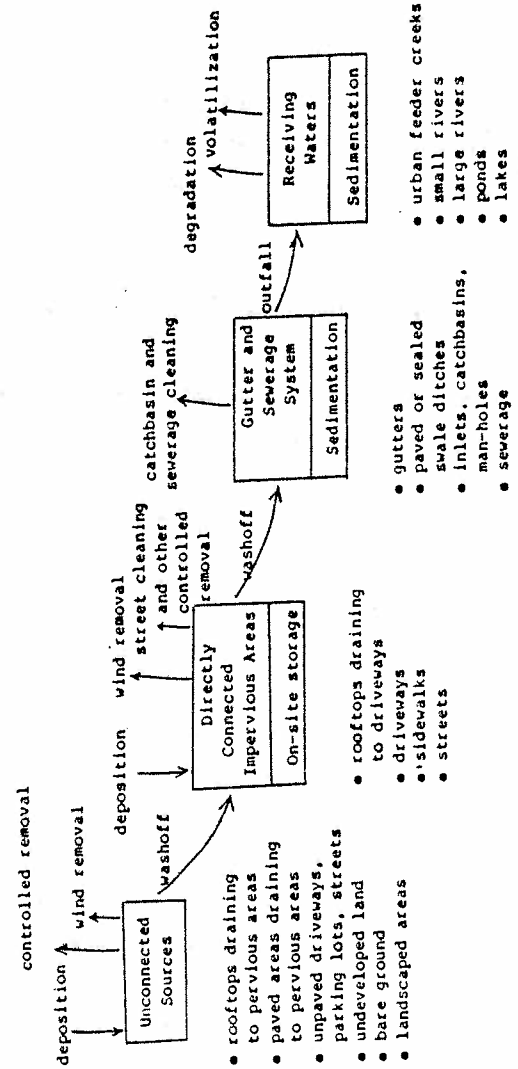


Figure 3.2 Pollutant Depositions and Removals at Source Areas

while other sources of pollutants are specific to the activities conducted on the areas. As examples, the ground surfaces of unpaved equipment or material storage areas can become contaminated by spills and debris, while undeveloped land remaining relatively unspoiled by activities can still contribute solids, organics, and nutrients, if eroded. Atmospheric deposition, deposition from activities on paved surfaces (auto traffic, material storage, etc.), and the erosion of material from upland unconnected areas are the major sources of pollutants in urban areas.

The washoff of debris and soil is dependent on the energy of the rain and the properties of the material removed. Pollutants are also removed from the source areas by wind, litter pickup, or other clean-up activities. The runoff flows and pollutants from the source areas directly enter the drainage system, or drain over pervious or impervious areas that will eventually be connected to the drainage system. Sewerage system sedimentation and cleaning may also affect the ultimate discharges at the outfall. In-stream physical, biological, and chemical processes affect the pollutants after they are discharged.

It is helpful to know when the different source areas become "active". If pervious source areas are not contributing flows or pollutants, then the prediction of urban runoff conditions is much simplified. In many cases, pervious areas are not active except for rains greater than about 5 or 10 mm. For rains of less depth, almost all flows and pollutants originate from impervious surfaces. In the upper mid-west, about 85 percent of all rains are less than 15 mm in depth. These events also generate about 70 percent of the total annual urban runoff volume. Rains of less than about 3 mm in depth account for about one-half of the number of rains, while rains less than about 12 mm in depth produce about one-half of the annual urban runoff volume. These are quite small rains, especially when compared to typical rains of concern in flooding and drainage studies (75 to 150 mm in depth).

The specific source areas that are of importance for different conditions varies widely, and modeling procedures that are sensitive to source contributions as a function of rain characteristics are needed.

B. Source Area Modeling Concept

Particulate pollutant contributions of source areas can be modeled by assuming the following mass balance relationship (with typical units):

L = sum of ($A_iQ_iP_iW_iD_i$), for i to n total source areas, where

- L is the total discharge of a specific pollutant at the outfall (mg),
- A is the source area in the drainage basin (ha),
- Q is the total quantity of source area limited particulates (kg/ha),
- P is the pollutant strength of the source area particulates (mg pollutant/kg particulate),
- W is the washoff fraction of the source area particulates, and
- D is the delivery yield of the washed-off source area particulates to the outfall.

The mass balance for filterable pollutants is much simpler, being the sum of the products of source area filterable pollutant concentrations and flows.

The Q parameter is applicable for "source limited" areas, where there is only a specific amount of pollutants available. For most rains, very few areas qualify as source limited. Relatively clean and smooth paved areas may be source limited during very large rains. For some areas it is impossible to be source limited (such as erosion products from pervious areas). The washoff of pollutants from impervious areas, for example, is usually limited by the energy of the rain and by armoring from overlying debris, and not by the absolute presence of pollutants. There is usually a substantial amount of pollutants left on most surfaces after rains.

Mass balance calculations must consider each source area, pollutant, and rain separately. The different pollutant deposition and removal processes and the runoff generation processes are quite different for each area and typically change for different rain characteristics. After each source area response is determined, the overall outfall discharges can be determined by adding the separate source area responses, after considering the drainage system and outfall processes affecting the discharges.

Most pollutants are only associated with a few source areas. As an example, sediment usually originates from streets (including ice control), vacant land, and construction sites; heavy metals originate mostly from roads and parking lots; and nutrients originate mostly from rain, litter, landscaped areas, and vacant lots. Most control measures are restricted to specific areas. For example, street cleaners can only operate on streets and parking lots, while runoff treatment at the outfall can control discharges from all source areas. Very few control measures are expected to be highly effective, but many are partially effective. Infiltration practices are the only controls that can effectively reduce filterable pollutants and flows, while sedimentation controls are restricted to reduction of particulate pollutants. It usually requires a variety of controls and control locations to achieve the desired urban runoff program benefits.

Careful application of the various controls to those source areas where they are most effective may result in an acceptable urban runoff control program.

Therefore, an understanding of the importance of the different source areas for a variety of conditions is necessary.

SECTION 4

BASIC STRUCTURE OF URBAN HYDROLOGY

A. Introduction

This section presents a large-scale overview of the information on basic urban hydrology which was obtained from the test watersheds. The test watershed outfall hydrology data were evaluated using exploratory cluster and principle component analyses, followed by quantitative stepwise and linear regression analyses. These analyses provided a description of the basic structure of urban hydrology, involving the interactions between various independent rain characteristics and dependent runoff characteristics. The information supported the potential use of a simplified approach of predicting runoff responses from urban watersheds during common, small rains most applicable for water quality studies. This simple structure is the basis of a more detailed and general

runoff volume prediction model developed in the next several sections.

B. Outfall Hydrology Observations for Test Watersheds

As part of this research, an extensive urban runoff quantity and quality monitoring effort was conducted in the Emery (industrial) and Thistledowns (residential and commercial) test watersheds of the Humber River basin in Toronto, Ontario from May 1983 through March 1984. The monitoring program included sampling and analyzing rain runoff, snowmelt runoff, warm weather baseflow, and cold weather baseflow at the two watershed outfalls in addition to sampling and analyzing many samples from source areas (as particulates and as sheetflows) during both warm and cold weather. This dissertation examines some of this information in detail; all of the data were summarized in a report prepared for the Ontario Ministry of the Environment (Pitt and McLean 1986).

The warm weather rain and outfall hydrology data obtained during the monitoring program are summarized in Tables 4.1 and 4.2. All rain and outfall flow

Table 4.1 Observed Emery Rain and Runoff Characteristics

Storm Number	Rain Start Date	Total Rain (mm)	Rain Dur. (hrs)	Ave. Rain Int. (mm/hr)	Peak 5-min. Rain Int. (mm/hr)	Preceeding Dry Period (days)	Total Discharge (mm)	Runoff Dur. (hrs)	Ave. Discharge (L/sec)	Peak 5-min. Discharge (L/sec)	Ave. Discharge (L/sec-ha)	Peak 5-min. Discharge (L/sec-ha)	Lag Time to Start of Runoff (Min)	Runoff Coef. (Rv)	Runoff/Rain Duration (ratio)	Peak/Ave. Discharge (ratio)
		Rain Characteristics (Independent Variables)					Runoff Characteristics (Dependent Variables)							Calculated Ratios		
7	5/14/83	11.0	5.75	1.91	21	6	3.2	5.03	241	1254	1.6	8.1	45	0.29	1.01	5.1
8	5/22	2.75	4.2	0.65	6	0	1.14	4.9	102	275	0.66	1.8	NA[1]	0.41	1.17	2.7
9	5/22	2.00	0.17	11.76	15	0.3	0.60	2.0	105	285	0.68	1.9	0	0.34	10.5	2.7
8 & 9	5/22	4.75	10.67	0.45	15	0	2.0	11.67	75	285	0.49	1.9	NA	0.42	1.09	3.0
10a	5/25	3.25	1.25	2.60	12	3	0.60	1.5	175	360	1.1	2.3	5	0.18	1.20	2.1
10b	5/25	3.00	1.0	3.00	15	0.2	0.89	3.17	124	500	0.81	3.0	30	0.30	3.17	4.7
10	5/25	6.25	6.1	1.02	15	3	1.63	0.67	82	500	0.53	3.0	5	0.26	1.42	7.3
11	5/29	14.00	2.6	5.38	12	4	4.27	3.92	470	960	3.1	6.2	15	0.31	1.51	2.0
12c	5/31	5.50	0.25	22.00	39	2	1.38	1.75	347	1060	2.3	6.9	15	0.25	7.0	3.1
12	5/31	8.50	6.4	1.32	39	0.3	3.92	11.75	146	1200	0.95	7.0	NA	0.46	1.03	0.2
13a	6/3	0.75	0.33	2.27	3	3	0.13	1.0	59	80	0.30	0.51	10	0.17	3.00	1.4
13b	6/3	2.25	1.42	1.58	3	0.1	0.51	2.5	90	140	0.50	0.91	20	0.23	1.76	1.6
13	6/3	3.00	3.25	0.92	3	3	0.73	4.5	71	140	0.46	0.94	10	0.24	1.38	2.0
14	6/5	1.00	0.42	2.38	3	1.2	0.17	1.6	47	85	0.31	0.55	30	0.17	3.0	1.8
15	6/6	11.00	5.83	1.89	6	1.1	2.99	6.25	210	470	1.4	3.1	30	0.27	1.07	2.2
16a	6/17	4.00	0.33	12.12	21	9	0.63	1.42	196	600	1.3	3.9	10	0.16	4.3	3.4
16b	6/17	3.75	0.33	11.36	24	0.2	0.26	1.25	93	165	0.60	1.1	10	0.07	3.0	1.8
16	6/17	8.00	4.7	1.70	24	9	1.03	5.58	81	600	0.53	3.9	10	0.13	1.19	7.4
17	6/27	9.00	16.7	0.54	3	10	1.11	16.5	30	45	0.20	0.29	NA	0.12	0.99	1.5

Table 4.1 Observed Emery Rain and Runoff Characteristics (Continued)

Storm Number	Rain Start Date	Total Rain (mm)	Rain Dur. (hrs)	Ave. Rain Int. (mm/hr)	Peak 5-min. Rain Int. (mm/hr)	Preceeding Dry Period (days)	Total Discharge (mm)	Runoff Dur. (hrs)	Ave. Discharge (L/sec)	Peak 5-min. Discharge (L/sec)	Ave. Discharge (L/sec-ha)	Peak 5-min. Discharge (L/sec-ha)	Lag Time to Start of Runoff (Min)	Runoff Coef. (Rv)	Runoff/Rain Duration (ratio)	Peak/Ave. Discharge (ratio)
		Rain Characteristics (Independent Variables)					Runoff Characteristics (Dependent Variables)							Calculated Ratios		
18	6/30	2.50	0.33	7.50	12	3	0.16	1.25	56	160	0.36	1.0	5	0.06	3.0	2.0
19	7/1	2.00	0.17	11.76	15	0.9	0.17	1.08	69	315	0.45	2.0	5	0.09	6.4	4.6
20	7/4	6.75	3.17	2.13	12	3	0.64	4.30	65	240	0.42	1.6	5	0.09	1.38	3.7
21	7/21	2.25	0.25	9.00	1.5	17	0.22	1.17	84	315	0.55	2.0	40	0.10	4.7	3.8
23	7/30	7.00	NA	NA	NA	9	0.64	3.42	82	540	0.53	3.5	NA	0.09	NA	6.6
24	7/31	2.00	0.66	3.03	12	0.5	0.25	6.25	18	120	0.12	0.70	25	0.13	9.5	6.7
26a	8/3	2.25	0.33	6.82	15	1.3	0.39	2.0	86	200	0.56	1.0	10	0.17	6.0	3.2
26b	8/4	2.75	5.33	0.52	6	0.1	0.01	5.33	66	180	0.43	1.2	20	0.29	1.00	2.7
26	8/3	5.00	4.92	1.02	15	1.3	1.24	0.5	64	280	0.42	1.0	10	0.25	1.73	4.4
27	8/8	10.50	1.5	12.33	57	5	2.52	2.75	403	1220	2.6	7.9	5	0.14	1.83	3.0
28a	8/11	8.50	4.75	1.79	6	3	1.26	4.83	114	360	0.74	2.3	50	0.15	1.02	3.2
28b	8/11	9.75	5.17	1.89	9	0.3	2.75	6.25	193	410	1.3	2.7	15	0.20	1.21	2.1
28	8/11	18.25	13.58	1.34	9	3	4.40	14.08	140	410	0.91	2.7	50	0.25	1.04	2.9
29a	8/22	0.75	0.42	1.79	3	10	0.10	0.92	48	60	0.31	0.39	15	0.13	2.19	1.3
29b	8/22	2.00	0.75	2.67	12	0.1	0.15	0.67	101	175	0.66	1.1	20	0.08	0.89	1.7
29c	8/22	20.25	3.17	6.39	27	0.1	4.95	3.75	579	1200	3.0	7.8	15	0.24	1.10	2.1
29	8/22	23.50	6.83	3.44	27	10	5.42	7.42	320	1200	2.1	7.0	15	0.23	1.09	3.0
30	8/27	1.25	0.5	2.50	6	5	0.73	1.17	276	920	1.0	6.0	10	0.58	2.34	3.3
31a	8/30	1.25	0.33	3.79	6	3	0.064	0.92	30	40	0.20	0.26	25	0.05	2.78	1.3
31b	8/30	1.75	0.67	2.61	6	0.2	0.25	2.0	55	110	0.36	0.71	5	0.14	3.0	2.0
31c	8/30	2.75	0.67	4.10	6	0.1	0.41	2.33	70	240	0.51	1.6	25	0.15	3.5	3.1
31d	8/30	1.00	0.75	1.33	3	0.2	0.36	2.75	58	150	0.38	0.75	45	0.36	3.5	2.6
31	8/30	7.00	16.2	0.43	6	3	1.50	10.5	36	240	0.23	1.6	25	0.21	1.14	6.7

Table 4.1 Observed Emery Rain and Runoff Characteristics (Continued)

Storm Number	Rain Start Date	Rain Characteristics (Independent Variables): Total Rain (mm)	Rain Dur. (hrs)	Ave. Rain Int. (mm/hr)	Peak 5-min. Rain Int. (mm/hr)	Preceding Dry Period (days)	Runoff Characteristics (Dependent Variables): Total Discharge (mm)	Runoff Dur. (hrs)	Ave. Discharge (l/sec)	Peak 5-min. Discharge (l/sec)	Ave. Discharge (l/sec-ha)	Peak 5-min. Discharge (l/sec-ha)	Lag Time to Start of Runoff (Min)	Calculated Ratios: Runoff Coef. (Rv)	Runoff/Rain Duration (ratio)	Peak/Ave. Discharge (ratio)
33b	9/16	11.50	2.75	4.10	15	0.06	4.0	4.0	430	1100	2.0	7.1	75	0.35	1.5	2.5
33	9/16	25.25	12.33	2.05	15	10	7.7	12.5	270	1100	1.0	7.1	NA	0.30	1	4.1
34	9/18	8.00	1.9	4.21	30	1.6	1.7	2.1	356	1140	2.3	7.4	90	0.21	1.1	3.2
35	9/21	12.75	4.5	2.83	12	3	3.7	5.3	310	710	2.0	4.6	20	0.29	1.2	2.3
37	9/25	2.25	3.2	0.70	3	3	0.27	2.5	47	75	0.31	0.49	75	0.12	0.8	1.6
38a	10/3	2.00	0.9	2.22	3	0	0.30	2.2	59	115	0.30	0.75	25	0.15	2.4	1.9
38b	10/3	1.00	0.3	3.33	6	0.1	0.22	1.5	64	115	0.42	0.75	10	0.22	5.8	1.8
38	10/3	3.00	2.7	1.11	6	8	0.51	3.7	61	115	0.40	0.75	25	0.17	1.4	1.9
39	10/4	18.25	0.6	30.42	63	0.6	3.87	2.0	607	1270	3.9	8.2	0	0.21	4.7	2.1
40	10/5	4.00	2.3	1.74	6	1.0	0.73	2.4	133	200	0.86	1.3	45	0.18	1.0	1.5
41	10/8	7.75	4.4	1.76	6	3	1.58	4.2	166	400	1.1	2.6	75	0.20	0.95	2.4
42	10/12	15.25	6.4	2.39	15	4	6.93	7.0	390	600	2.5	4.0	60	0.45	1.2	1.7
43a	10/13	11.25	4.5	2.50	26	1.2	4.14	5.9	310	670	2.0	4.4	10	0.37	1.3	2.2
43b	10/14	1.50	0.5	3.00	7.5	0.4	0.33	1.2	125	205	0.81	1.3	5	0.22	2.4	1.6
43	10/13	13.00	14.3	0.91	26	1.2	5.64	14.9	166	670	1.1	4.4	10	0.43	1.0	4.0
44	10/16	2.50	1.4	1.39	3	3	0.40	2.25	70	160	0.51	1.8	NA	0.16	1.6	2.1
45	10/23	14.75	11.6	1.27	6	7	4.53	11.67	170	520	1.1	3.4	55	0.31	1.0	3.1
46	11/2	9.00	7.0	1.15	6	10	1.55	0.17	83	250	0.54	1.6	20	0.17	1.0	3.0
Number of obs.		60	59	59	59	60	60	60	60	60	60	60	54	60	59	60
Minimum		0.75	0.17	0.43	1.5	0.06	0.06	0.67	10	40	0.43	1.5	0	0.05	0.0	1.3
Maximum		25.25	16.7	30.4	63	17	7.7	10.5	607	1210	30.4	63	90	0.50	16.5	0.2
Mean		6.9	3.0	4.1	14	3.6	1.0	4.9	161	462	1.0	3.0	24	0.23	2.5	3.1

(*) NA = not available

Table 4.2 Observed Thistledowns Rain and Runoff Characteristics

Storm Number	Rain Start Date	Rain Characteristics (Independent Variables): Total Rain (mm)	Rain Dur. (hrs)	Ave. Rain Int. (mm/hr)	Peak 5-min. Rain Int. (mm/hr)	Preceding Dry Period (days)	Runoff Characteristics (Dependent Variables): Total Discharge (mm)	Runoff Dur. (hrs)	Ave. Discharge (l/sec)	Peak 5-min. Discharge (l/sec)	Ave. Discharge (l/sec-ha)	Peak 5-min. Discharge (l/sec-ha)	Lag Time to Start of Runoff (Min)	Calculated Ratios: Runoff Coef. (Rv)	Runoff/Rain Duration (ratio)	Peak/Ave. Discharge (ratio)
21b	7/24/83	1.25	0.03	1.58	6	7	0.19	2.3	10.4	24	0.27	0.62	15	0.15	2.7	2.3
26a	8/3	2.25	0.66	3.41	15	1.3	0.054	1.0	6.0	13	0.18	0.33	30	0.02	1.5	1.9
26b	8/4	2.75	1.92	1.43	6	0.1	0.31	2.2	17	36	0.44	0.93	40	0.11	1.1	2.1
26	8/3	5.00	4.92	1.02	15	1.3	0.43	5.3	10	36	0.26	0.93	30	0.09	1.1	3.6
27	8/8	17.25	1.42	12.15	58	5	6.33	1.2	680	1800	17	46	15	0.37	0.85	2.6
28a	8/11	0.50	4.33	1.96	6	3	0.66	4.0	22	58	0.57	1.5	5	0.10	1.1	2.6
28b	8/11	9.50	6.08	1.56	9	0.1	1.89	5.9	40	190	1.0	4.9	10	0.20	0.97	4.0
28	8/11	18.00	13.9	1.29	9	3	2.88	13.8	26	190	0.67	4.9	5	0.16	0.99	7.3
29a	8/22	2.00	2.0	0.69	3	11	0.43	3.6	15	20	0.39	0.72	10	0.22	1.2	1.9
29b	8/22	20.00	3.1	6.45	27	0.03	4.38	3.1	127	600	3.3	15	5	0.22	1.0	4.7
29	8/22	22.00	6.0	3.24	27	11	4.8	7.9	76	600	2.0	15	5	0.22	1.2	7.9
31a	8/30	1.25	0.3	4.17	6	3	0.14	1.5	12	23	0.31	0.59	10	0.11	5.0	1.9
31b	8/30	1.75	0.7	2.50	6	0.2	0.17	1.4	16	33	0.41	0.85	50	0.10	2.0	2.1
31c	8/30	2.75	0.7	3.93	6	0.1	0.26	1.4	23	67	0.59	1.7	30	0.09	2.0	2.9
31d	8/30	1.00	0.8	1.25	3	0.3	0.12	1.0	8	11	0.21	0.20	35	0.12	2.3	1.4
31	8/30	7.00	16.2	0.43	6	3	1.15	17.7	8	67	0.21	1.7	10	0.16	1.1	8.4
32	9/6	0.75	0.2	3.75	6	7	0.10	0.9	24	58	0.62	1.3	0	0.24	4.5	2.1
33a	9/16	14.00	0.2	1.71	6	10	2.19	9.6	29	190	0.75	4.9	20	0.16	1.2	6.6
33b	9/16	6.00	1.3	4.62	15	0.06	1.18	1.3	111	340	2.9	8.7	25	0.20	1.0	3.1

Table 4.2 Observed Thistledowns Rain and Runoff Characteristics (Continued)

Storm Number	Rain Start Date	Total Rain (mm)	Rain Dur. (hrs)	Ave. Rain Int. (mm/hr)	Peak 5-min. Rain Int. (mm/hr)	Pre-ceeding Dry Period (days)	Total Discharge (mm)	Runoff Dur. (hrs)	Ave. Discharge (l/sec)	Peak 5-min. Discharge (l/sec)	Ave. Discharge (l/sec-ha)	Peak 5-min. Discharge (l/sec-ha)	Lag Time to Start of Runoff (Min)	Runoff Coef. (Rv)	Runoff/Rain Duration (ratio)	Peak/Ave. Discharge (ratio)
		Rain Characteristics (Independent Variables)					Runoff Characteristics (Dependent Variables)							Calculated Ratios		
33c	9/16	5.75	1.2	4.79	10	0.01	1.74	1.4	156	410	4.0	11	10	0.30	1.2	2.6
33	9/16	25.75	12.3	2.09	18	10	5.10	12.4	52	410	1.3	11	20	0.20	1.0	7.9
34	9/10	0.00	1.9	4.21	30	1.6	1.62	1.3	162	770	4.2	20	40	0.20	0.7	4.0
35	9/21	12.75	4.5	2.83	12	3	2.53	4.8	66	290	1.7	7.5	20	0.20	1.1	4.4
37	9/25	2.25	3.2	0.70	3	3	0.20	2.9	9	14	0.23	0.36	70	0.09	0.9	1.0
38	10/3	3.00	2.8	1.07	6	0	0.20	3.6	9	37	0.23	0.95	25	0.09	1.4	4.1
39	10/4	10.25	0.6	30.42	63	0.6	1.40	0.6	247	1010	6.4	26	25	0.08	1.3	4.1
40	10/5	4.00	2.3	1.74	6	1.0	0.52	3.3	20	43	0.51	1.1	40	0.13	1.4	2.2
41	10/8	7.50	3.7	2.03	6	3	1.29	3.9	41	164	1.1	4.1	50	0.17	1.1	3.9
42	10/12	15.00	5.8	2.59	12	4	4.31	0.9	78	240	2.0	6.2	30	0.29	1.2	3.1
43a	10/13	11.25	4.6	2.45	21	1.2	2.16	5.2	52	270	1.3	6.9	20	0.19	1.1	5.2
43b	10/14	1.50	0.5	3.00	NA	0.4	0.25	1.3	24	40	0.62	1.0	25	0.17	2.6	1.7
43	10/13	13.00	14.4	0.90	21	1.2	2.82	15.3	23	270	0.59	0.9	20	0.22	1.1	11.7
44	10/16	2.5	1.4	1.79	3	3	0.31	2.9	14	43	0.36	1.1	40	0.12	2.1	3.1
45	10/23	14.75	11.7	1.26	6	3	3.25	10.3	40	194	1.0	5.0	90	0.22	0.9	4.9
46	11/2	9.00	7.0	1.15	6	10	1.36	0.0	19	47	0.49	1.2	30	0.15	1.1	2.5
Number of obs.		36	35	35	34	35	35	35	35	35	35	35	35	35	35	35
Minimum		0.75	0.20	0.43	3.0	0.01	0.05	0.0	9	14	0.17	0.28	0	0.02	0.2	1.6
Maximum		25.75	16.2	30.4	63	11	6.3	17.2	600	1800	17.5	46.3	90	0.37	5.0	11.7
Mean		0.5	4.4	3.4	14	3.5	1.6	4.9	65	250	1.7	6.3	26	0.17	1.5	3.9

(*) NA = not available

information were recorded on magnetic tape data loggers during the monitoring period, resulting in a complete and continuous description of all rains and resultant flows. The recorded data were plotted to obtain continuous outfall hydrographs with simultaneous rain intensity records.

The events listed in these two tables represent distinct hydrologic events in that the outfall hydrographs returned to near baseflow conditions at the end of each event. Some events were separated by only about 1-1/2 hours, or less, while the typical interevent period (preceding dry period) was three to four days. The longest interevent period observed was 17 days.

The total number of events monitored at Emery was 60, while 35 separate events were monitored at Thistledowns. The warm weather Emery monitoring period lasted from May 14 through November 2, 1983, while the Thistledowns monitoring did not begin until July 1983. Typical total rain depth totals during this monitoring period were about 310 mm, occurring on about 56 days. The events monitored represented almost all of the runoff that occurred at each outfall during the monitoring period. Only about three rains and a total rain depth of about 4 mm were missed in the monitoring program.

Figure 4.1 shows two accumulative distribution plots for rain counts and runoff volume associated with different rain depths. The total rain depth was less than 3 mm for more than one half of the rain events that occurred during this monitoring period, while the median accumulative runoff volume was associated with rain depths of about 12 mm. The largest observed depth of rainfall was only 25 mm. Rains greater than 100 mm in depth would occur only once every several years. These very large rains would contribute large runoff volumes and quantities of stormwater pollutants per event, but because they are rare, they contribute only a few percent of the annual pollutant and flow discharges.

These distributions stressed the importance of small rains when considering water quality. Pitt and McLean (1986) found that almost 90 percent of the annual stormwater discharges of most pollutants were associated with rains less than 20 mm in depth, and 25 percent of the annual stormwater pollutant discharges were associated with rains less than 8 mm in depth. Heavy metal discharges were even more associated with

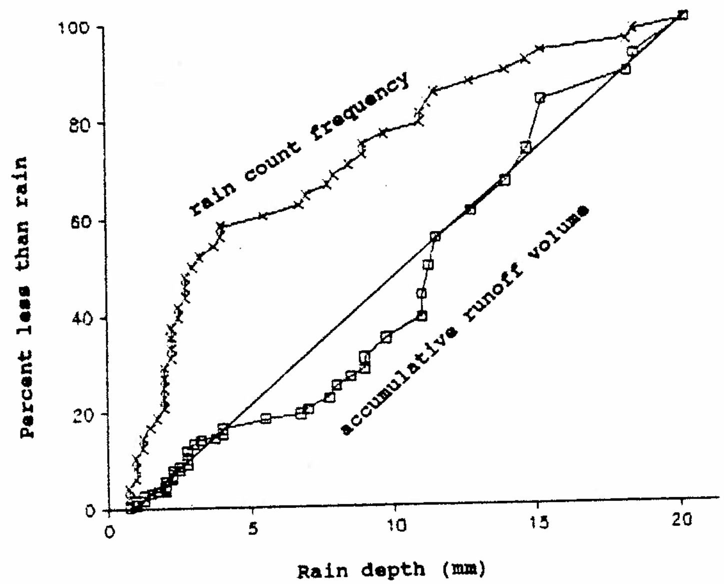


Figure 4.1 Rainfall Distribution at Emery

small rains, while nutrient discharges were associated with slightly larger rains. Therefore, when considering water quality problems, the common, small rains were much more important than the rare, larger rains most commonly associated with flooding and drainage problems.

The observed rain durations (and average intensities) were obviously influenced by the way events were defined. The continuous hydrographs obtained during this research enabled separating the storms by actual watershed responses; storms "ended" when the receding limb of the hydrographs approached the pre-storm baseflow rate. If the definition of the interevent period was changed, then the rain distributions shown on Figure 4.1 would also have changed. In the absence of continuous hydrograph information, many urban runoff studies have used arbitrary interevent periods of at least six hours of no rain between events to separate distinct runoff events (the usual time needed for the hydrographs to recede to baseflow conditions for typical storm sewered urban drainage areas).

Area-normalized runoff responses were needed for this research to examine the effects that different

land surface covers had on outfall runoff responses. Tables 4.1 and 4.2 therefore express runoff volume as depth, and discharges as both volume per unit time (L/sec) and volume per unit time per unit area (L/sec-ha). The unit area normalized discharge values were used in developing the urban runoff structural relationships presented later in this section.

C. Cluster Analysis of Test Watershed Outfall Hydrology Data

Cluster analysis was used to determine the overall inter-relationships of the rain and outfall runoff data for the Emery and Thistledowns test watersheds. Cluster analysis is a multivariate procedure used for detecting groupings in data. It is similar to discriminant analysis in that it results in subgroupings of data. Discriminant analysis, however, starts with known subgroupings while cluster analysis forms the subgroupings. Dillon and Goldstein (1984) stated that the goal of many cluster analysis applications is to arrive at clusters that display small within-cluster variations compared to between-cluster variations.

Anderberg (1973) found that cluster analysis may be used to reveal structure and relations in the data: it is a tool of discovery.

Cluster analysis is most commonly used to group data observations, such as in identifying closely related source areas of urban runoff pollutants. However, the analysis described in this subsection grouped variables. Anderberg (1973) found that hierarchal clustering with normalized values were needed for clustering variables. The result of this hierarchal analysis is a tree diagram (or dendogram), showing the linkages of each group of data as a joining of branches. The root of the tree is the linkage of all of the data into one cluster. Moving from the branches towards the root depicts increasing aggregation of the data into larger clusters (Lebart et al. 1984).

The choice of a different distance scale in a cluster analysis can have significant effects on results (Dillon and Goldstein 1984). If the common Euclidean distance method is used, which is not scale invariant, a change in scale can result in vastly different trees. If a variable measure is changed from feet to inches, for example, completely different cluster groupings can result. The use of Pearson

correlation coefficient distances, in contrast, standardize each variable and force the weightings between variables to be equal.

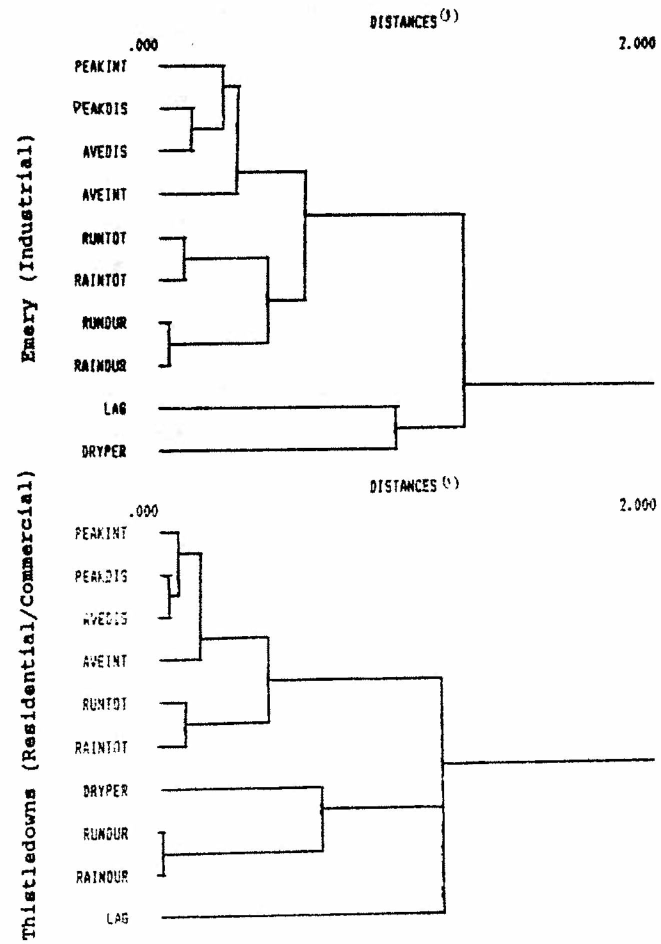
Single linkage, or nearest neighbor, is the oldest and simplest linkage procedure (Massart and Kaufman 1983). In this method, the distance between two clusters is equal to the distance between the two closest data observations in the two clusters. To identify clusters, the single linkage method first finds the two closest data observations (objects) as the first cluster. It then searches for the next closest object to this first cluster. If it is closer to either object in this first cluster than to a fourth object, it is combined as part of the first cluster. If the third object is closer to a fourth object than either object in the first cluster, then the third and fourth objects form the second cluster. This process continues until all of the objects belong to a cluster. This single linkage procedure is incapable of delineating poorly separated clusters (Anderberg 1973). Hence it is a conservative linkage procedure: any two objects in a single cluster will be more similar to each other than to any other object in another cluster.

Cluster analysis can be used as an aid for the application of other techniques. Massart and Kaufman (1983) found that one of the best combinations of classification methods was the use of clustering in conjunction with principal components. This dissertation section uses these two procedures as preliminary steps in developing regression models describing urban runoff outfall hydrology conditions.

The cluster program used in this analysis was SYSTAT - The System for Statistics, Version 3 (1986), from SYSTAT, Inc., Evanston, Il. Tables 4.1 and 4.2 contain the input data used in the cluster analysis. SYSTAT offered many analytical options that were tried on these data. The most suitable results were obtained by using Pearson correlation coefficients to standardize the distance measurements and single linkage to form the clusters. All distances were computed using pairwise deletion of missing values. SYSTAT uses standard hierarchal amalgamation algorithms described by Hartigan (1975) and tree ordering algorithms described by Gruvaeus and Wainer (1972).

Figure 4.2 contains the two tree diagrams resulting from the final cluster analysis of the Emery

Figure 4.2 Cluster Analysis (Tree Diagram) for Basic Urban Hydrology Structure



(1) Distance metric is 1-Pearson correlation coefficient (normalized) and the linkage method is nearest neighbor

and Thistledowns data. The tree branches are labeled with the following variable names:

RAINTOT, total depth of rain, mm

RAINDUR, rain duration, hours

AVEINT, average rain intensity, mm/hr

PEAKINT, peak 5-minute rain intensity, mm/hr

DRYPER, preceding dry period without rain, days

RUNTOT, total runoff volume, mm

RUNDUR, runoff duration, hours

AVEDIS, average runoff discharge rate, L/sec-ha

PEAKDIS, peak 5-minute runoff discharge rate, L/sec-ha

LAG, lag time between start of rain and start of runoff, hours

The trees are printed so that the most similar variables (objects) are adjacent to each other.

The tree diagram is an excellent exploratory data analysis technique and contains much information concerning the inter-relations between the variables. Of special interest in these tree diagrams is the relative repeatability of the basic structure of urban hydrology for the two very different test watersheds.

Simple variable relationships can be easily recognized. The following list shows the Pearson distances between closely spaced variables:

	Emery	Thistledowns
RUNTOT - RAINTOT	0.09	0.09
RUNDUR - RAINDUR	0.04	0.01
AVEDIS - PEAKDIS	0.12	0.04
PEAKDIS - PEAKINT - AVEDIS	0.25	0.07

These distances are all very small and indicate significant cluster groupings. All of the variables do not form a single large group until the Pearson distance exceeds 1.2 for Emery and 1.1 for Thistledowns. The above simple linkage distances are therefore very small relative to the linkage distance for the complete group. In all of these cases, except for PEAKINT - PEAKDIS - AVEDIS, simple two-way relationships exist.

Complex relationships can also be seen from the tree diagrams. At both sites, PEAKINT, PEAKDIS, AVEDIS, and AVEINT are relatively closely related (having a Pearson correlation cluster separation of only 0.3 at

Emery and 0.16 at Thistledowns). Variables poorly related to other variables are also shown on these tree diagrams. LAG at both test watersheds and DRYPER (especially at Emery) are examples of variables having little relationship with other variables (they enter the main cluster near "last").

Other procedures used to identify inter-relationships include correlation matrices. Table 4.3 contains correlation matrices for the data presented in Tables 4.1 and 4.2 for comparison. The following list shows the correlation coefficients between the same simple (two member) clusters that were identified above:

	Emery	Thistledowns
RUNTOT - RAINTOT	0.906	0.903
RUNDUR - RAINDUR	0.965	0.989
AVEDIS - PEAKDIS	0.849	0.946
PEAKDIS - PEAKINT	0.748	0.917

With the exception of the last two pairs for Emery, these are very high correlation coefficients and demonstrate the similarities in simple structure that

Table 4.3 Correlation Matrices for Basic Urban Hydrology Structure

Emery (Industrial)

	RAINTOT	RAINDUR	AVEINT	PEAKINT	DRYPER	RUNTOT	RUNDUR	AVEDIS	PEAKDIS	LAG
RAINTOT	1.000									
RAINDUR	0.533	1.000								
AVEINT	0.138	-0.387	1.000							
PEAKINT	0.512	-0.039	0.675	1.000						
DRYPER	0.169	0.273	-0.096	-0.132	1.000					
RUNTOT	0.906	0.562	0.087	0.405	0.075	1.000				
RUNDUR	0.501	0.965	-0.348	0.035	0.184	0.556	1.000			
AVEDIS	0.709	-0.013	0.480	0.654	-0.095	0.680	-0.026	1.000		
PEAKDIS	0.729	0.129	0.372	0.748	0.041	0.699	0.150	0.849	1.000	
LAG	0.135	0.220	-0.292	-0.217	0.052	0.205	0.134	0.098	0.107	1.000

Thistledowns (Residential/Commercial)

	RAINTOT	RAINDUR	AVEINT	PEAKINT	DRYPER	RUNTOT	RUNDUR	AVEDIS	PEAKDIS	LAG
RAINTOT	1.000									
RAINDUR	0.553	1.000								
AVEINT	0.321	-0.295	1.000							
PEAKINT	0.564	-0.104	0.827	1.000						
DRYPER	0.281	0.308	-0.190	-0.122	1.000					
RUNTOT	0.903	0.448	0.187	0.551	0.283	1.000				
RUNDUR	0.508	0.989	-0.322	-0.148	0.337	0.402	1.000			
AVEDIS	0.398	-0.178	0.593	0.817	-0.037	0.585	-0.227	1.000		
PEAKDIS	0.600	-0.051	0.659	0.917	0.009	0.702	-0.166	0.946	1.00	
LAG	-0.192	-0.037	-0.114	-0.202	-0.122	-0.184	-0.094	-0.138	-0.173	1.000

can be found by either correlation matrices or cluster analysis. However, the more complex relationships (greater than two member clusters) cannot be identified from the correlation matrices.

D. Principle Component Analysis of Test Watershed Outfall Hydrology Data

Principal component analysis is another technique that can be used to simplify large masses of data and to help identify the basic structure of the system under study. As stated earlier, it complements cluster analysis. Its goal is to group variables into a few principal components that can explain most of the variance observed in the data.

Principal component analysis also starts with a correlation (or covariance) matrix of the variables and simplifies the relationships by grouping the variables into principal components. For studies needing only simple two-way relationships to be identified, simply scanning for "high" coefficients in the matrix is adequate. However, the number of coefficients can become large and simple correlation coefficients cannot

indicate complex relationships. As shown on Table 4.3, this research examined ten variables, resulting in 45 correlation coefficients to be evaluated. These correlation matrices resulted in the same simple close relationships identified by the cluster analysis, but they were unable to directly show the more complex relationships evident from the cluster analysis. The principal component analysis was conducted to verify the overall urban hydrology structure identified earlier and to rank the relationships between the important variables.

Principal components allows the most "significant" variables that account for most of the data variability to be readily identified. The component loadings are the ordinary product-moment correlation of each variable and the component (Dillon and Goldstein 1984). The SYSTAT program that was used in this analysis allowed sorting of the variables according to their component loadings which simplified identifying the structure of the system.

The use of a covariance matrix eliminates differences associated with the means of the variables, leaving the variations about the means to be evaluated. The variables used in a covariance principal component analysis should not have grossly different variances. If the variances differ greatly, then the first few principal components will be heavily influenced by the variables having the larger variances (Dillon and Goldstein 1984). Dillon and Goldstein still generally recommend this transformation because of its beneficial effects of eliminating the scale influences of the variables.

Rotation of the principal components is used to help achieve simple structure. As an example, if the variables are located relatively large distances from the principal component axes, they will all have relatively low component loadings. Rotation of the principal component axes can move some of the variables closer to the rotated axes, while moving others further away, more efficiently separating the variable component loadings for the different principal components. SYSTAT allows several rotation options. The rotation used in the final principal component analysis for this research was the varimax rotation. This rotation method is quite popular and rotates component axes so that the variation of the squared component loadings for a given component is made large (Dillon and Goldstein 1984).

The issue of factor scores (and their subsequent analysis) has become controversial in the decision of selecting principal component or common factor analyses (SYSTAT documentation). Factor scores are calculated for each observation after the principal component analysis. These scores give the location of each observation in relation to the principal components. Principal component analysis allows the factor scores to be directly calculated, while common factor analysis (including maximum likelihood factor analysis) allows only the factor scores to be estimated (the "indeterminacy problem"). Principal component analysis takes the given data and attempts to determine the dimensions defining the total variance. Some authors consider principal component analysis as a type of common factor analysis, but others want it kept separate. Principal component analysis simply defines the basic dimensions of the data and makes no assumptions about common factors; while common factor analysis can assume the number or character of the dimensions and, under certain conditions, these assumptions can be tested (Dillon and Goldstein 1984). However, SYSTAT and Dillon and Goldstein all report that principal component and common factor solutions for real data rarely significantly differ.

Table 4.4 shows the resultant rotated component loadings and the variance explained for each principal component for the hydrology data presented in Tables 4.1 and 4.2. Covariance matrices were used to eliminate the effects of the different scales used for the variables, the loadings were sorted and pairwise deletion was used to eliminate missing data sets. Varimax rotation was also used for the analysis summarized in this table. SYSTAT allowed many different options for principal component analysis. Other alternative analytical procedures tried included correlation matrices; listwise deletion; and no, equamax, and quartimax rotations.

The striking feature of this table is the similarity of the basic structure for the two completely different test watersheds and the reasonable groupings of the variables. The first component for both sites explains about 40 percent of the total variability and is made up of "total" (RUNTOT and RAINTOT) and "intensity" (AVEDIS, PEAKDIS, AVEINT, and PEAKINT) variables. The second principal components explain another 27 percent of the variation at both

Table 4.4 Principle Components for Basic Hydrology Structure (Covariance Correlation with Pairwise Deletion and Varimax Rotation)

Emery (Industrial)

Rotated Loadings:	1 (Totals and Intensities)	Major Descriptors: 2 (Duration)	3 (Lag Period)	4 (Dry Period)
AVEDIS	0.941	-0.064	-0.094	-0.071
PEAKDIS	0.930	0.084	-0.051	0.044
PEAKINT	0.811	-0.050	0.408	-0.105
RAINTOT	0.782	0.513	-0.084	0.122
RUNTOT	0.722	0.576	-0.194	0.003
AVEINT	0.561	-0.466	0.482	0.012
RUNDUR	0.062	0.968	-0.005	0.053
RAINDUR	0.054	[illegible]	-0.107	0.147
LAG	0.087	0.057	-0.926	0.025
DRYPER	-0.023	0.128	0.028	[illegible]
Variance explained by Varimax Rotated Loadings:				
% in principal component:	38.7%	27.0	13.3	10.4
Accumulation %:	38.7%	65.7	79.0	89.4

Thistledowns (Residential/Commercial)

Rotated Loadings:	1 (Totals and Intensities)	Major Descriptors: 2 (Duration)	3 (Lag Period)	4 (Dry Period)
PEAKDIS	0.975	0.050	-0.043	0.071
PEAKINT	0.953	0.010	-0.113	-0.134
AVEDIS	0.913	-0.108	-0.011	0.080
AVEINT	0.774	-0.217	-0.092	-0.257
RUNTOT	0.666	0.577	-0.034	0.302
RAINTOT	0.610	0.684	-0.077	0.198
RAINDUR	-0.122	0.969	0.006	0.092
RUNDUR	-0.179	0.951	-0.060	0.111
LAG	-0.117	-0.057	[illegible]	-0.068
DRYPER	-0.050	0.193	-0.072	[illegible]
Variance explained by Varimax Rotated Loadings:				
% in principal component:	41.7%	27.4	10.1	11.4
Accumulative %:	41.7%	69.1	79.2	90.6

(1) Only the first four principal components are shown.

sites and include the "duration" variables (RAINDUR and RUNDUR). The third components explain about 10 to 13 percent of the variability and are almost exclusively comprised of the "lag" variable. The fourth components also explain about 10 percent of the variability and are comprised of one variable alone, the dry period between rains.

The most interesting (and complicated) relationships are found mostly in the first component (the dependent variables being runoff total, average discharge rate, and peak discharge rate). The relationship between runoff duration and rain duration and the lag time between start of rain and start of runoff are expected to vary for different sites depending on their sizes and drainage efficiencies (related to "time of concentration" in flooding studies). The dry period before the rain is readily available from climatic records and can be analyzed to determine seasonal trends for different geographical areas. By developing general prediction models for runoff total, average discharge rate, and peak discharge rate, about 40 percent of the total urban hydrology structure variation can be explained. By also determining the simpler relationship between rain

duration and runoff duration, and the "unrelated" lag time, about 80 percent of the variance can be explained. Adding the dry period allows about 90 percent of the total variability to be explained. The use of these five independent rain variables and five dependent runoff variables can result in comprehensive descriptions of urban hydrology at the test sites.

The following subsections build upon these structural groupings of variables in developing prediction models for all of the dependent runoff variables. This structural information will also be used in later sections to develop runoff prediction models that are more generally applicable to other locations.

E. Stepwise Regression Analysis of Test Watershed Outfall Hydrology Data

Stepwise regression analysis of the data presented in Tables 4.1 and 4.2 was conducted to extract information about the structure of urban hydrology using SYSTAT.

Tables 4.5 and 4.6 show the results of the stepwise regressions for the dependent runoff parameters RUNTOT, RUNDUR, AVEDIS, and PEAKDIS. The alpha-to-enter and the alpha-to-remove values were both 0.15 for this analysis. These tables show the order in which the independent rain parameters entered the "model" and the accumulative regression coefficients (R^2) at each step. Note, however, that the ordering of parameters were of little theoretical significance. The complete list of parameters recommended by the stepwise regression (with these alpha values) were then evaluated in separate models using multivariate general linear regression analysis in the next subsection. Tables 4.5 and 4.6 also show the model coefficients, the standard errors, and the two-tail probability values for significance for each included independent variable from these general linear regressions.

This stepwise regression analysis resulted in models which may be more complex than warranted, even though all of the model parameters were found to be typically very significant. As an example, the models describing RUNDUR gained very little in further explaining the data variance (as reflected in R^2, the multiple correlation coefficient squared, or the ratio

Table 4.5 Summarized Results of Stepwise Regression Analyses for Basic Hydrology Relationships - Emery[1]

Dependent Variables:

Independent Variables:	RUNTOT Order Selected	Accum. R^2	Model Coef. (2)	Standard error(3)	P (2-tail)(3)	RUNDUR Order Selected	Accum. R^2	Model Coef.	Standard Error	P (2-tail)
RAINTOT	1	0.83	0.29	0.02	<0.001	3	0.94	-0.07	0.03	0.033
RAINDUR	-	-	-	-	-	1	0.93	1.04	0.04	<0.001
AVEINT	2	0.84	-0.06	0.02	0.018	-	-	-	-	-
PEAKINT	-	-	-	-	-	2	0.94	0.04	0.01	0.005
DRYPER	3	0.85	-0.04	0.03	0.13	4	0.95	-0.06	0.04	0.12
Constant term	-	-	0.11	0.18	0.57	-	-	1.11	0.27	<0.001

Independent Variables:	AVEDIS Order Selected	Accum. R^2	Model Coef. (3)	Standard error(3)	P (2-tail)(3)	PEAKDIS Order Selected	Accum. R^2	Model Coef.	Standard Error	P (2-tail)
RAINTOT	1	0.51	0.14	0.01	<0.001	2	0.72	0.24	0.04	<0.001
RAINDUR	2	0.72	-0.09	0.02	<0.001	3	0.74	-0.15	0.06	0.014
AVEINT	3	0.75	0.03	0.01	0.024	4	0.75	-0.09	0.05	0.088
PEAKINT	-	-	-	-	-	1	0.56	0.12	0.02	<0.001
DRYPER	4	0.76	-0.03	0.02	0.11	-	-	-	-	-
Constant term	-	-	0.40	0.12	0.001	-	-	0.57	0.34	0.10

(2) Alpha-to-enter and alpha-to-remove are 0.15.

(3) Calculated variable coefficients, standard errors, and probability values were determined from separate multivariate general linear regression analysis using constant terms and the independent variables suggested by the stepwise regression analyses.

Table 4.6 Summarized Results of Stepwise Regression Analyses for Basic Hydrology Relationships - Thistledowns(1)

Dependent Variables:

Independent Variables:	RUNTOT Order Selected	RUNTOT Accum. R^2	RUNTOT Model Coef. (2)	RUNTOT Standard error(2)	RUNTOT P (2-tail)(2)	RUNDUR Order Selected	RUNDUR Accum. R^2	RUNDUR Model Coef.	RUNDUR Standard Error	RUNDUR P (2-tail)
RAINTOT	1	0.81	0.22	0.02	<0.001	2	0.94	-0.04	0.02	0.033
RAINDUR	-	-	-	-	-	1	0.90	1.01	0.03	<0.001
AVEINT	-	-	-	-	-	-	-	-	-	-
PEAKINT	-	-	-	-	-	-	-	-	-	-
DRYPER	-	-	-	-	-	3	0.98	0.05	0.03	0.095
Constant term	-	-	-0.21	0.20	0.29	-	-	0.62	0.10	0.002

Independent Variables:	AVEDIS Order Selected	AVEDIS Accum. R^2	AVEDIS Model Coef. (2)	AVEDIS Standard error(2)	AVEDIS P (2-tail)(2)	PEAKDIS Order Selected	PEAKDIS Accum. R^2	PEAKDIS Model Coef.	PEAKDIS Standard Error	PEAKDIS P (2-tail)
RAINTOT	-	-	-	-	-	-	-	-	-	-
RAINDUR	3	0.72	-0.13	0.07	0.079	-	-	-	-	-
AVEINT	2	0.69	-0.23	0.11	0.043	2	0.87	-0.55	0.20	0.011
PEAKINT	1	0.67	0.25	0.04	<0.001	1	0.84	0.78	0.08	<0.001
DRYPER	-	-	-	-	-	-	-	-	-	-
Constant term	-	-	-0.32	0.54	0.56	-	-	-2.33	0.05	0.010

(1) Alpha-to-enter and alpha-to-remove are 0.15.

(2) Calculated variable coefficients, standard errors, and probability values were determined from separate multivariate general linear regression analyses using constant terms and the independent variables suggested by the stepwise regression analyses.

of the sum of squares due to the regression to the sum of squares about the mean) by adding additional parameters beyond RAINDUR. The Thistledowns AVEDIS and PEAKDIS models also had very small increases in R^2 values after the first parameters were added. However, the Emery models for AVEDIS and PEAKDIS require the first two parameters to obtain accumulative R^2 values greater than 0.7, but adding additional model parameters only increased the R^2 values to about 0.75.

The stepwise regression models revealed structural relationships that were quite similar to the relationships found in the earlier exploratory analyses, but the procedure resulted in excessively complex relationships, considering the limited added benefits of the additional variables. The exploratory analyses, however, did not produce quantitative relationships. The stepwise regression models were used to guide the development of simple linear regression models, as will be described in the next subsection.

F. Linear Regression Analysis to Test Structural Urban Hydrology Models for the Test Watersheds.

Candidate models were developed to describe the dependent runoff parameters as various linear functions of the independent rainfall parameters for the two test watersheds. These models were based on the urban hydrology structure revealed in the previously described exploratory cluster and principal components analysis, the stepwise regression analysis, and past experience of urban hydrology research. The simple regression routines of SYSTAT's comprehensive multivariate general linear hypothesis (MGLH) package were used for this analysis.

The remaining tables in this section summarize the selected models for RUNTOT, PEAKDIS, AVEDIS, RUNDUR, and LAG. The RUNTOT models in Table 4.7 are similar for both test watersheds, with the intercept (constant term) varying by about 10 percent and the slope (RAINTOT coefficient) varying by about 25 percent. The standard errors for the constant terms are relatively large and the corresponding probability values indicate little significance. The error and probability terms for the RAINTOT coefficients, on the other hand, are

Table 4.7 Selected Runoff Total Volume Models

Emery (Industrial)

RUNTOT = -0.186 + 0.279 (RAINTOT)

60 observations
multiple R^2 = 0.82

Variable	Coef.	Standard Error	P(2-tail)
Constant	-0.186	0.158	0.244
RAINTOT	0.279	0.017	< 0.001

Thistledowns (Residential/Commercial)

RUNTOT = -0.213 + 0.217 (RAINTOT)

35 observations
multiple R^2 = 0.81

Variable	Coef.	Standard Error	P(2-tail)
Constant	-0.213	0.197	0.287
RAINTOT	0.271	0.018	< 0.001

both relatively small and are very significant. SYSTAT also tests the models for redundant variables (evident if the calculated eigenvalues are close to zero, or if high correlations are evident in the correlation matrix of the regression coefficients). For this model, the only possible redundancy would be between the constant term and the RAINTOT coefficient, which was not indicated.

Comprehensive residual analyses were conducted for all candidate models to test the satisfactory applicability of the regression assumptions requiring constant variance of the residuals: the residuals were normally distributed; they had zero mean; and they did not display any trends with time or trends with dependent or independent model parameters, (Draper and Smith 1981). Figure 4.3 shows a summary of an example residual analysis for the Emery RUNTOT model. This model does not fully explain the variance presented in the observed runoff volume data so the residuals are relatively large (the R^2 values are only about 0.8 for these models). The residuals follow a normal probability distribution quite well, and show no obvious trends with sampling sequence or predicted runoff volume.

Figure 4.3 Runoff Total Volume Model Residual Behavior - Emery

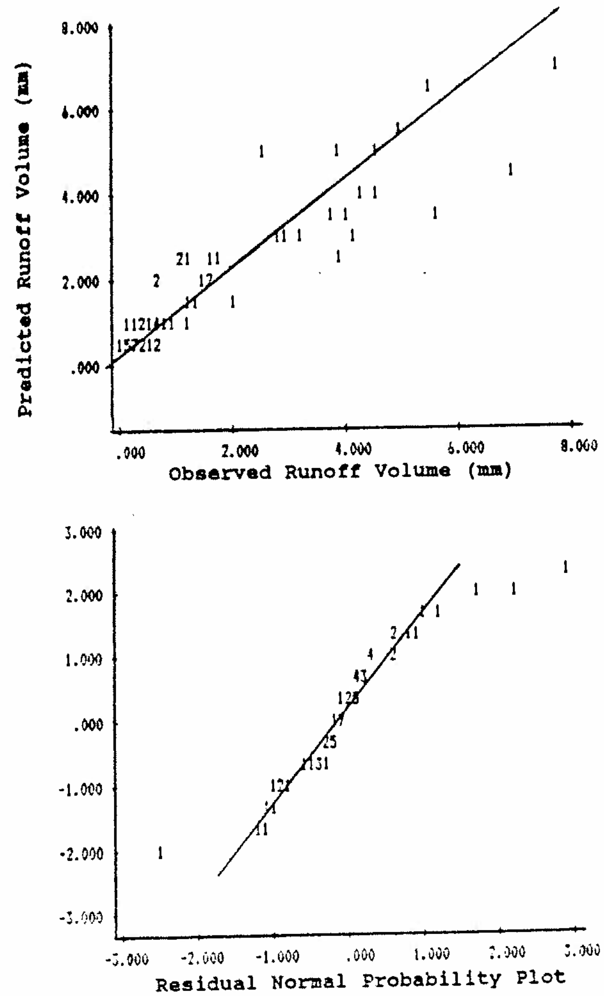
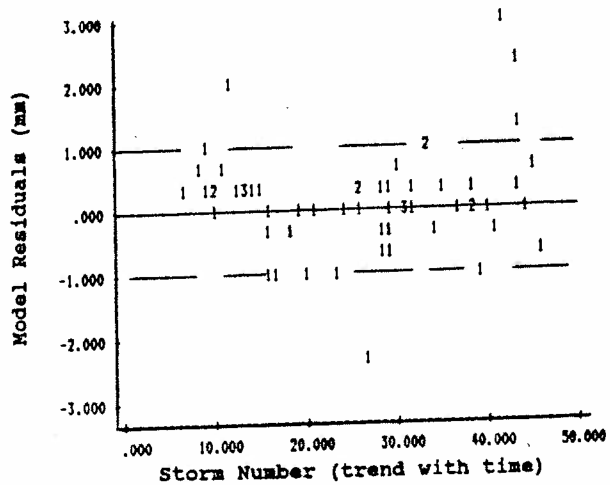


Figure 4.3 Runoff Total Volume Model Residual Behavior - Emery (Continued)



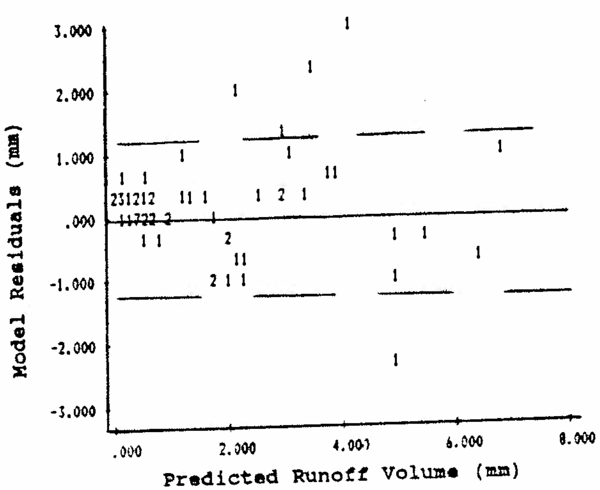


Table 4.8 summarizes the models for PEAKDIS, the 5-minute peak runoff discharge rate. Two models are presented for Emery, one having both PEAKINT and RAINTOT as independent variables, and the second having only PEAKINT as an independent variable. The analysis did not indicate redundancy with the more complex Emery PEAKDIS model, but a regression relationship between PEAKINT and PEAKDIS was desired to compare to the Thistledowns model and was more theoretically defensible. The earlier exploratory analyses indicated that the Emery peak discharge variable was more complex than the Thistledowns peak discharge variable so both model types are summarized here. The constant and PEAKINT coefficient terms are very different for the two test watersheds, indicating substantial site specific influences on the peak discharge rate characteristic that was not nearly as evident for the runoff volume characteristic.

Table 4.9 summarizes the models for AVEDIS, the event averaged runoff discharge rate. Two models are shown for each test watershed; the first models are the "simplified" models derived from the stepwise regression analysis while the second models simply relate AVEDIS with AVEINT. Even though the more complex

Table 4.8 Selected Peak Discharge Rate (5-minute) Models

Emery (Industrial)

PEAKDIS = 0.296 + 0.102 (PEAKINT) + 0.190 (RAINTOT)

59 observations
multiple R^2 = 0.71

Variable	Coef.	Standard Error	P(2-tail)
Constant	0.296	0.285	0.303
PEAKINT	0.102	0.016	<0.001
RAINTOT	0.190	0.033	<0.001

PEAKDIS = 0.958 + 0.150 (PEAKINT)

59 observations
multiple R^2 = 0.55

Variable	Coef.	Standard Error	P(2-tail)
Constant	0.958	0.325	0.005
PEAKINT	0.150	0.018	< 0.001

Thistledowns (Residential/Commercial)

PEAKDIS = -1.892 + 0.610 (PEAKINT)

34 observations
multiple R^2 = 0.84

Variable	Coef.	Standard Error	P(2-tail)
Constant	-1.892	0.912	0.046
PEAKINT	0.610	0.047	<0.001

Table 4.9 Selected Average Discharge Rate Models

Emery (Industrial)

AVEDIS = 0.471 + 0.148 (RAINTOT) - 0.117 (RAINDUR)

59 observations
multiple R^2 = 0.72

Variable	Coef.	Standard Error	P(2-tail)
Constant	0.471	0.099	<0.001
RAINTOT	0.148	0.012	<0.001
RAINDUR	-0.117	0.018	<0.001

AVEDIS = 0.710 + 0.083 (AVEINT)

59 observations
multiple R^2 = 0.23

Variable	Coef.	Standard Error	P(2-tail)
Constant	0.710	0.133	<0.001
AVEINT	0.083	0.020	<0.001

Thistledowns (Residential/Commercial)

AVEDIS = -0.791 + 0.182 (PEAKINT)

34 observations
multiple R^2 = 0.67

Variable	Coef.	Standard Error	P(2-tail)
Constant	-0.791	0.441	0.082
PEAKINT	0.182	0.023	<0.001

AVEDIS = 0.457 + 0.354 (AVEINT)

35 observations
multiple R^2 = 0.33

Variable	Coef.	Standard Error	P(2-tail)
Constant	0.457	0.513	0.380
AVEINT	0.354	0.083	<0.001

models resulted in "better" regression coefficients, the simpler models were preferred. The exploratory structural analyses indicated complex relationships for average discharge rate, but the "significant" predictor variables suggested by the stepwise regression analysis were not similar to the "close" related variables identified in the exploratory analyses. Again, site specific conditions in the two test watersheds apparently greatly influenced the average discharge rate characteristic.

Figure 4.4 is a summary of the residual analysis for the complex AVEDIS model for Emery. In contrast to the earlier presented residual analysis summary for RUNTOT, this model had a poorer fit with the data and was more complex. The plots still indicated reasonably good residual behavior, however.

Table 4.10 summarizes the regression analysis for the RUNDUR models. These models showed very good fits with the data and were very simple and theoretically reasonable. In addition, the RAINDUR coefficients are almost identical for both test watersheds. These coefficients are very close to 1.0, indicating very direct relationships between RUNDUR and RAINDUR (as shown in the cluster and principal component analyses).

Figure 4.4 Average Discharge Rate Model Residual Behavior Emery: AVEDIS = f(constant, RAINTOT, RAINDUR)

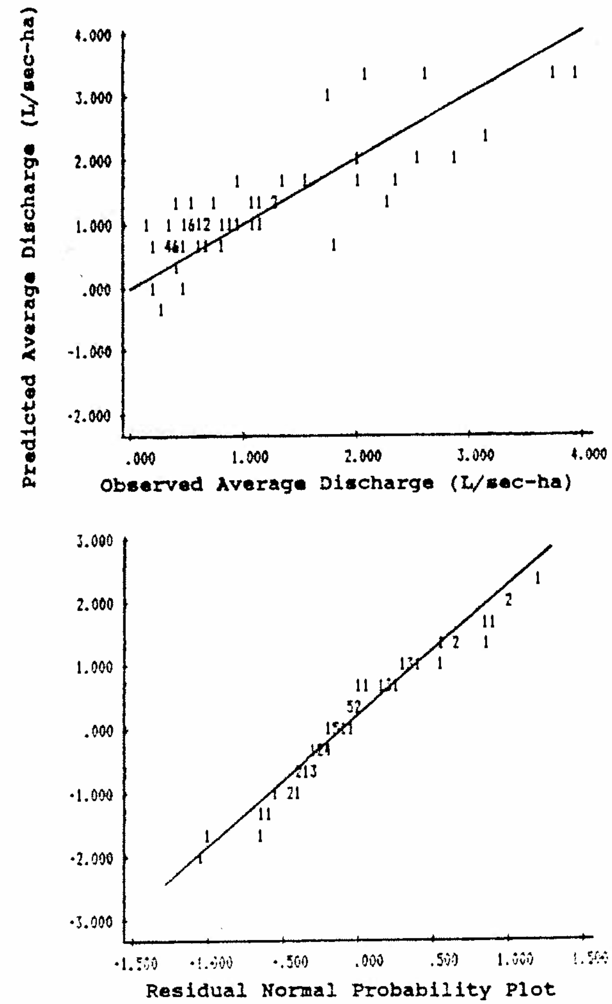
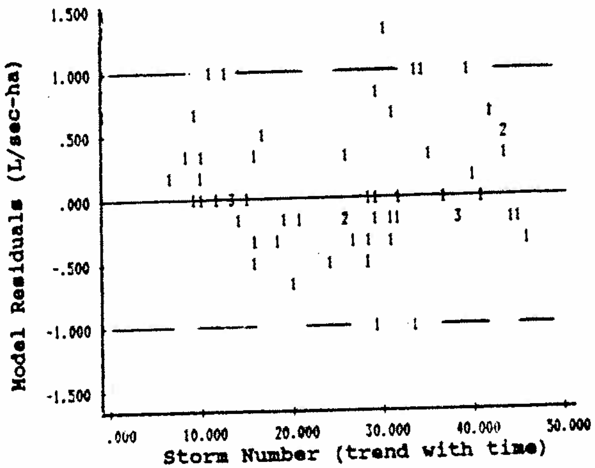


Figure 4.4 Average Discharge Rate Model Residual Behavior Emery (Continued)



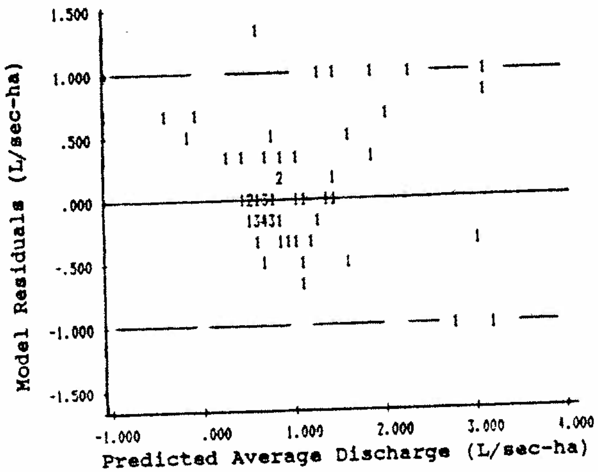


Table 4.10 Selected Runoff Duration Models

Emery (Industrial)

RUNOUR = 1.247 + 0.964 (RAINOUR)

59 observations
multiple R^2 = 0.93

Variable	Coef.	Standard Error	P(2-tail)
Constant	1.247	0.198	<0.001
RAINOUR	0.964	0.035	<0.001

Thistledowns (Residential/Commercial)

RUNOUR = 0.554 + 0.991 (RAINOUR)

35 observations
multiple R^2 = 0.98

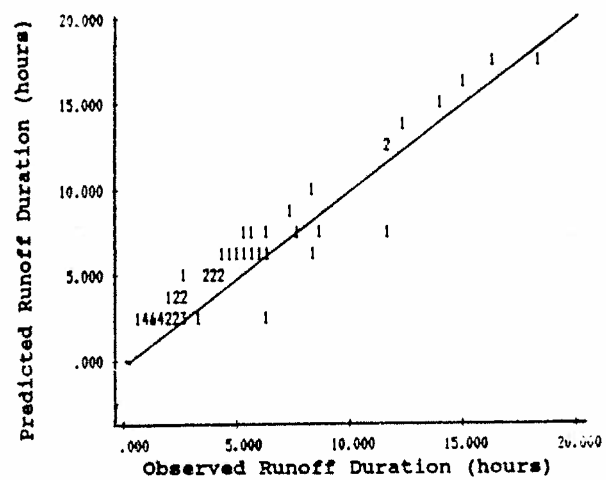
Variable	Coef.	Standard Error	P(2-tail)
Constant	0.554	0.157	0.001
RAINOUR	0.991	0.025	<0.001

The constant terms were quite different, with the Emery runoff durations being about 3/4 of an hour longer than the Thistledowns runoff durations. Drainage system characteristics and test watershed sizes were probably most responsible for these differences.

Figure 4.5 illustrates residual behavior for the simple and "good" Emery runoff duration model, in contrast to the simple but moderately good RUNTOT model and the complex and poor AVEDIS model presented earlier. Again, residual behavior appears to be satisfactory.

Table 4.11 summarizes two models for LAG, the lag period between the start of runoff and the start of rain. For both test watersheds, only a constant term was used to predict this variable. The resultant constants were very similar for both test watersheds, about 25 minutes, and the standard errors were relatively small, about 3 minutes. Tables 4.1 and 4.2 show ranges in lag times from almost zero to about 90 minutes for both watersheds. The cluster and principal component analyses did not show any close relationships of LAG with any independent variable and the stepwise regression analysis did not result in any meaningful and stable suggested list of recommended variables.

Figure 4.5 Runoff Duration Model Residual Behavior - Emery



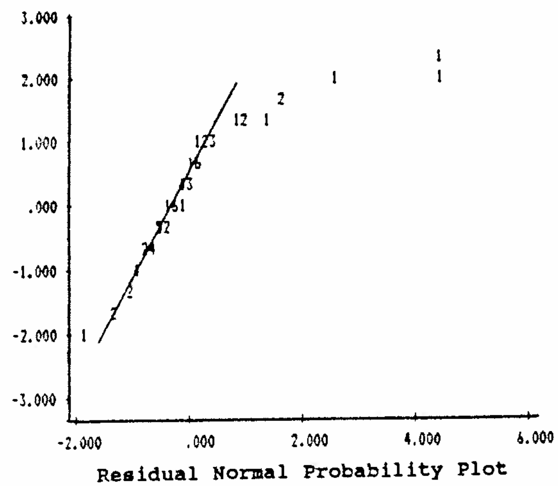
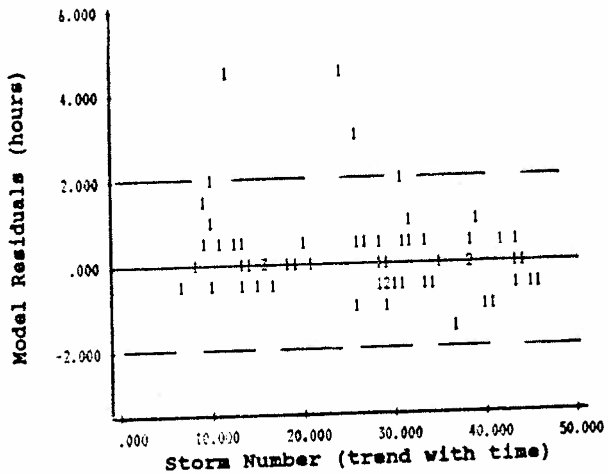


Figure 4.5 Runoff Duration Model Residual Behavior - Emery (Continued)



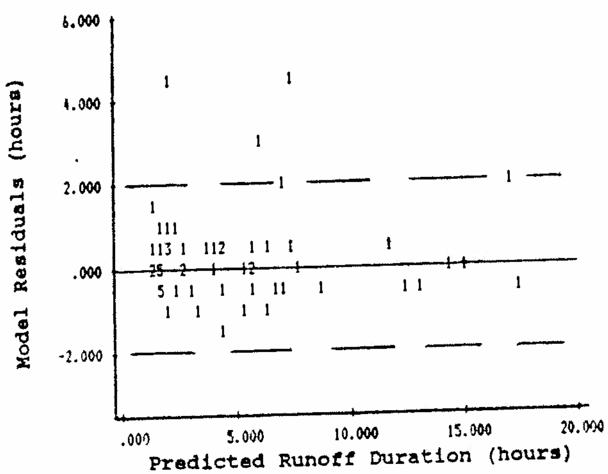


Table 4.11 Selected Lag Time Models

Emery (Industrial)

LAG = 24.2 minutes

53 observations
multiple R^2 not applicable

Variable	Coef.	Standard Error	P(2-tail)
Constant	24.2	3.0	<0.001

Thistledowns (Residential/Commercial)

LAG = 25.9 minutes

35 observations
multiple R^2 not applicable

Variable	Coef.	Standard Error	P(2-tail)
Constant	25.9	3.2	<0.001

G. Summary of Urban Hydrology Structure

The exploratory and quantitative regression analyses described in this section revealed a reasonable structure for the important urban hydrology parameters by indicating how they interrelated and were influenced by rain characteristics. In addition, this structure was reasonably repeatable between the different exploratory and regression analytical techniques and for two very different test watersheds. There were some obvious differences, but they could be generally explained by weaknesses or strengths in the techniques or by obvious site specific influences affecting time-dependent runoff variables. In general, the runoff parameters were most closely (and simply) related to their obvious rain parameter counterparts, especially for RUNTOT and RAINTOT and for RUNDUR and RAINDUR. The runoff rate parameters, PEAKDIS and AVEDIS, were more complex and appeared to be significantly related to site specific watershed characteristics. The LAG periods were found to be unrelated to any of the independent rain variables, so

constant (average) values were used which were surprisingly similar for both test watersheds.

Many of the structural characteristics found during this research conflicted with the parameter inter-relationships assumed for flood and drainage analyses. As is described in Appendix C, many of the urban hydrology models in use for water quality analyses were originally developed for predicting runoff response for very large rains. In water quality studies, the common (and therefore small) rains are of most concern. The most obvious structural conflict between the large event models and the structure identified by evaluating the common rains observed during this research was the simple relationship that runoff volume had with rain total. Most flooding analyses predict runoff volume from a combination of rain (both for the event of interest and antecedent conditions), soil, and drainage system characteristics, at a minimum. Total runoff volume should be the hydrology parameter of most interest in water quality studies, whereas peak runoff rate and time of concentration are the most important parameters in flooding analyses. Therefore, the hydrology prediction models used for these very different analyses should be different.

Water quality models should utilize the simple relationship between runoff and rain volumes (with a minimum of site specific land development information), while flooding models need to consider more detailed drainage, soil, and antecedent rain characteristics. Most importantly, water quality analyses should not be required to include more information than is necessary. Most water quality management decisions are not sensitive to instantaneous flow rates or rapid concentration changes (which are typically inaccurate and costly to predict). Seasonal pollutant yields and relatively long-term average concentration distributions are usually of most concern in water quality studies.

Unfortunately, the "models" examined in this section will be of little use outside of the two test watersheds. No information was used that would enable these relationships to be generally applicable to other urban areas. The purpose of this section was to explore the structural relationship of urban hydrology and to compare several different exploratory analytical methods to discover and independently verify this

structure. There was, however, a similarity in this structure between the two very different test watersheds. This similarity is the basis for the more general (transferable) quantitative analyses presented in the next sections.

The next several sections of this dissertation explore important runoff volume characteristics that can be used to develop a general prediction model suitable for water quality studies. This model is finally constructed and compared to runoff information collected over several years at eight locations in Milwaukee as an independent verification.

SECTION 5

SMALL-SCALE PAVED AREA HYDROLOGY TESTS

A. Introduction

This section presents the results of the controlled street runoff experiments conducted in Toronto, Ontario as part of this research. These experiments were conducted to determine the influencing environmental factors affecting runoff characteristics from small paved areas under a variety of rain and pavement conditions.

As discussed in the Key Concepts section and in Appendix C, runoff from paved areas can be responsible for most of the runoff observed in urban areas. However, common methods to predict runoff from paved areas have been typically developed as part of flooding models and are most appropriate for large rains. These models make simplifying assumptions regarding runoff losses for paved areas which do not affect significantly the runoff predictions for the large

events. These models can be suitably used when evaluating flooding problems. Unfortunately, the simplifications result in significant errors when the models are used to predict the runoff from paved areas during the common small events that are of most interest for water quality analyses. With inaccurate paved area runoff predictions, estimates of runoff (and pollutant) sources are also inaccurate. Knowing the runoff sources becomes very important when attempting to evaluate alternative runoff quantity and quality controls.

These small-scale paved area runoff experiments were conducted to carefully examine runoff losses from paved areas under controlled rain conditions for small rain volumes. This information was then used to develop a general runoff model for paved areas that was further expanded by using data from large paved parking areas and roofs. This section presents the results of the small-scale controlled tests, while the next section discusses the large-scale uncontrolled impervious area tests.

B. Small-Scale Test Data

A series of tests were conducted as a nested 2^3 factorial experiment (Box et al. 1978). The experimental variables examined were rain intensity, street texture, and street dirt loading. Experimental nesting was with time, so the additional variables of time of rain and time of runoff were also included in the experiment. Elapsed rain and runoff volumes, along with instantaneous runoff rates, were the main experimental measures. A series of particulate washoff observations were also obtained during these tests that were used to develop a particulate washoff model (as discussed later in Section 9).

Table 5.1 presents the site data along with the basic rain and runoff observations obtained during these tests. All tests were conducted for about two hours, with total rain volumes ranging from about 5 to 25 mm. The test code explanations follow.

Table 5.1 Controlled Street Sheetflow Hydrology Tests

Rough Streets							
HCR*		HDR*		LCR*		LDR*	
rain (mm)	runoff (mm)	rain (mm)	runoff (mm)	rain (mm)	runoff (mm)	rain (mm)	runoff (mm)
0.92	0.00	1.22	0.00	0.39	0.00	0.51	0.00
1.47	0.35	1.42	0.35	0.48	0.015	0.97	0.16
2.76	1.25	2.24	0.90	0.96	0.32	1.43	0.38
4.60	2.25	3.46	1.64	1.49	0.71	1.58	0.47
6.44	3.39	4.27	2.23	2.17	0.98	1.84	0.56
10.12	6.02	5.49	3.01	2.89	1.38	2.45	1.00
11.59	6.99	6.30	3.55	3.56	1.69	2.91	1.32
13.80	8.65	10.37	6.37	4.72	2.61	3.32	1.61
16.56	10.24	13.01	8.20	5.78	3.16	4.18	1.99
17.48	11.04	14.44	9.10			5.10	2.52
19.32	12.60	17.28	11.17			6.12	3.46
23.00	15.71	18.50	12.07				
		20.94	13.95				
		24.60	16.72				

test*	location	"rain" intensity (mm/hr)	street dirt load (g/m^2)	street texture (depression storage, mm)
HCR	3 Humberland Ct.	11.0	3.3	1.05
HDR	3 Humberland Ct.	12.2	12.8	1.05
LCR	2 Humberland Ct.	2.9	3.0	1.05
LDR	2 Humberland Ct.	3.1	11.2	1.05

test*	date	time	weather conditions	temperature
HCR	08/18/83	1300-1500	warm and overcast	22-23°C
HDR	08/16/83	1400-1600	warm and sunny	23-25°C
LCR	08/23/83	1400-1600	hot and sunny (1)	18-23°C
LDR	08/19/83	1030-1230	hot and sunny (1)	17-20°C

(1) Test sites were shaded to simulate overcast conditions.

Table 5.1 Controlled Street Sheetflow Hydrology Tests (Continued)

HCS and HDS* rain (mm)	HCS and HDS* runoff (mm)	Smooth Streets LCS* rain (mm)	Smooth Streets LCS* runoff (mm)	L(D)CS* rain (mm)	L(D)CS* runoff (mm)
0.40	0.00	0.26	0.00	0.38	0.00
1.00	0.29	0.41	0.026	0.48	0.019
2.40	1.17	0.67	0.15	0.75	0.20
4.21	2.44	1.28	0.53	1.29	0.43
6.01	3.64	1.80	1.00	1.93	0.80
10.22	6.77	2.93	1.69	3.32	1.61
12.42	8.38	4.06	2.46	5.47	3.13
18.23	13.11	4.98	3.15	7.19	4.02
20.43	14.93	6.52	4.18		
24.04	18.17				

test*	location	"rain" intensity (mm/hr)	street dirt load (g/m²)	street texture (depression storage, mm)
HDS	61 Bankfield	12.1	13.8	0.42
HCS	61 Bankfield	12.0	2.6	0.42
LCS	121 Alhart	3.1	2.3	0.29
L(D)CS	121 Alhart	3.2	2.4 (2)	0.29

test*	date	time	weather conditions	temperature
HDS	08/17/83	1300-1500	warm and cloudy	21-25°C
HCS	08/19/83	1330-1530	warm and overcast	22-27°C
LCS	08/26/83	1000-1200	warm and cloudy	19-21°C
L(D)CS	08/24/83	1300-1500	warm and sunny (3)	20-22°C

(2) "LDS" was not dirty as intended and was therefore a loading replicate with LCS.

(3) Test sites were shaded to simulate overcast conditions.

	rain intensity	street dirt	street texture
HCR	high	clean	rough
HDR	high	dirty	rough
LCR	light	clean	rough
LDR	light	dirty	rough
HDS	high	dirty	smooth
HCS	high	clean	smooth
LDS	light	dirty	smooth
LCS	light	clean	smooth

Table 5.1 shows the specific experimental levels that each variable was held to during each test. Unfortunately, the streets during the LDS test were not as dirty as anticipated and was actually a replicate with the LCS tests. The experimental analyses were modified to indicate these unanticipated duplicate observations.

C. Runoff Loss Observations During the Small-Scale Runoff Tests

General Descriptive Plots

Figure 5.1 is a simple plot of these rainfall and runoff observations. The data are seen to group along two general lines, one for smooth streets and the other for rough streets. The other variables (rain intensity and street dirt loading) had little apparent effect.

This figure also illustrates how the volumetric runoff coefficients (Rv) varied with rain volume. The Rv values were about 0.5 (half of the rain occurred as runoff) for rain volumes less than about 5 mm, while the Rv values increased to about 0.7 to 0.75 for the largest rains evaluated. The initial runoff losses were about 1 mm. The variable runoff losses (all runoff losses after the initial losses were satisfied) were about 50 percent of the rain volume for the small rains and about 25 to 30 percent of the rain volume for the larger rains. These losses were substantially greater than the losses typically assumed for paved areas by the flood hydrology models.

Figure 5.2 is an example plot illustrating the runoff loss relationships for the HCR experiment, while

Figure 5.1 Rainfall-Runoff Responses for Small-Scale Tests

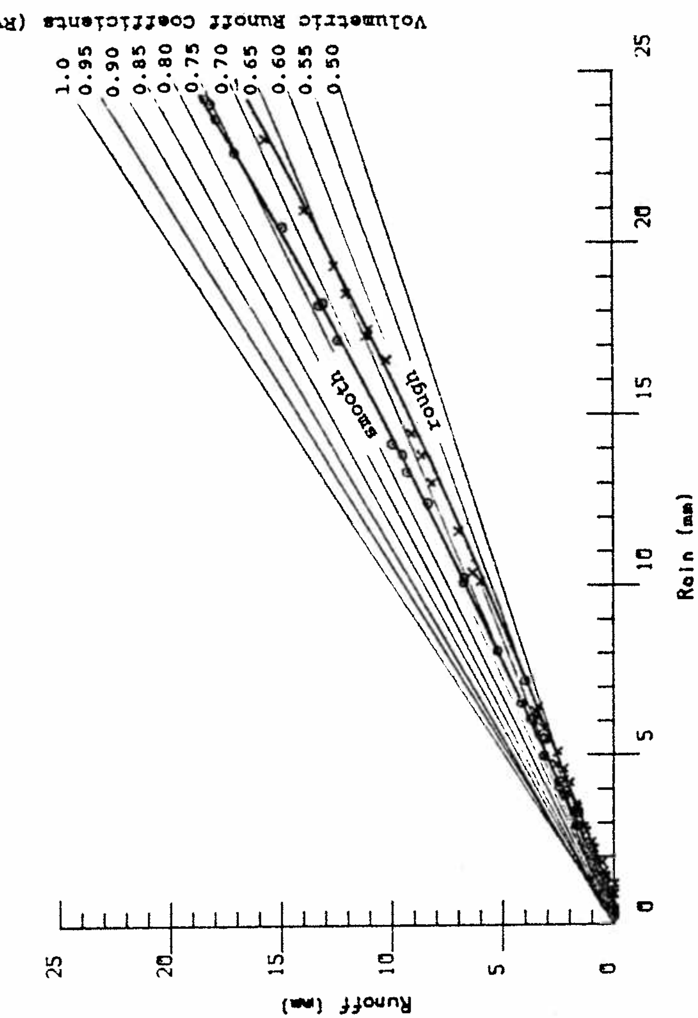


Figure 5.2 Rainfall-Runoff Plot Showing Loss Definitions—
Example for HCR Test

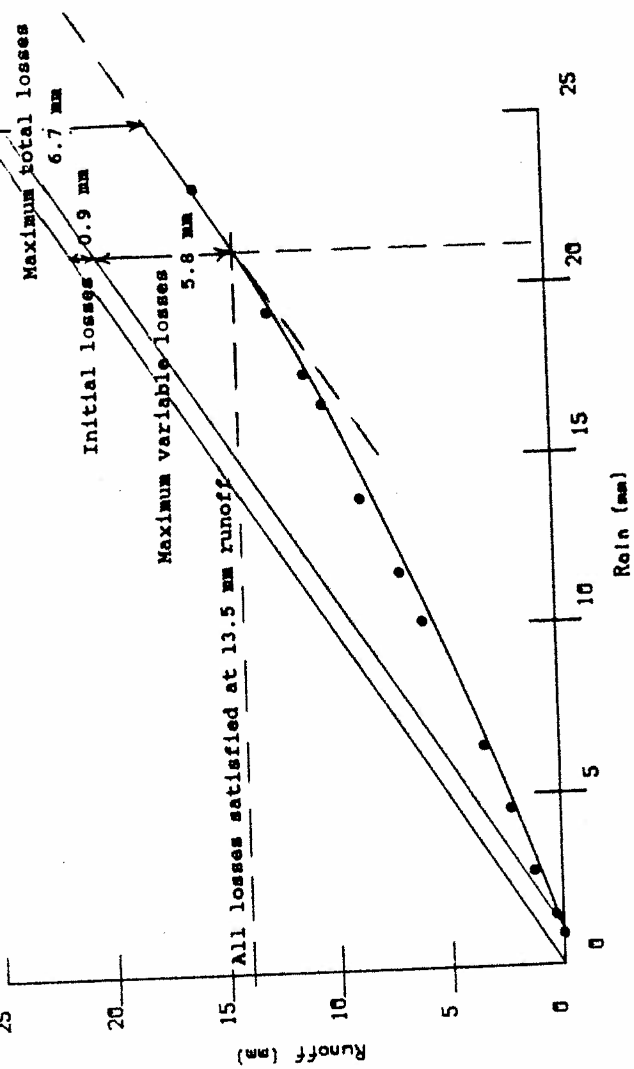


Table 5.2 summarizes the initial and variable losses for all of the tests. This table shows the initial losses and the time required for the rain to satisfy these initial losses (based on observations of runoff initiation). Runoff starts after these initial losses are satisfied. The variable losses are the total losses (rain minus runoff volume) after subtracting these initial losses.

<u>Factorial Test Calculation Descriptions</u>

The runoff tests were designed in a complete 2^3 factorial experiment to identify the significant variables affecting the observed runoff characteristics of each experiment. Box et al. (1978) described factorial experiments as being extremely useful for measuring the effects of one or more variables on a response. They also stated that factorial experimental designs are economical and easy to use and can provide a great deal of information. Factorial experimental analysis are a form of analysis of variance (ANOVA). The analysis of variance label is somewhat misleading, as the ANOVA procedure really compares means and not variances.

Table 5.2 Observed Initial and Variable Runoff Losses

HCR
Initial losses:

time needed to satisfy initial losses: 5.0 min.

rain volume needed to satisfy initial losses: 0.9 mm

Variable losses:

time since start of runoff (min)	rain vol. since start of runoff (mm)	variable runoff losses (mm)
0	0.0	0.0
3	0.6	0.2
10	1.8	0.6
20	3.7	1.4
30	5.5	2.1
50	9.2	3.2
58	10.7	3.7
70	12.9	4.2
85	15.6	5.4
90	16.6	5.5
100	18.4	5.0
120	22.1	6.4

HDR
Initial losses:

time needed to satisfy initial losses: 5.9 min.

rain volume needed to satisfy initial losses: 1.1 mm

Variable losses:

time since start of runoff (min)	rain vol. since start of runoff (mm)	variable runoff losses (mm)
0	0.0	0.0
1	0.2	-0.2
5	1.0	0.1
11	2.2	0.6
15	3.1	0.8
21	4.3	1.3
25	5.1	1.5
45	9.2	2.8
58	11.8	3.6
65	13.2	4.1
79	16.1	4.9
85	17.3	5.2
97	19.7	5.8
115	23.4	6.7

HCS and HDS
Initial losses:

time needed to satisfy initial losses: 2.0 min.

rain volume needed to satisfy initial losses: 0.4 mm

Variable losses:

time since start of runoff (min)	rain vol. since start of runoff (mm)	variable runoff losses (mm)
0	0.0	0.0
3	0.6	0.3
18	2.0	0.8
19	3.8	1.4
28	5.6	2.0
49	9.8	3.1
60	12.0	3.6
67	13.4	3.9
89	17.8	4.7
100	20.0	5.1
118	23.6	5.5

Table 5.2 Observed Initial and Variable Runoff Losses (Continued)

LCR
Initial losses:

time needed to satisfy initial losses: 8.3 min.

rain volume needed to satisfy initial losses: 0.4 mm

Variable losses:

time since start of runoff (min)	rain vol. since start of runoff (mm)	variable runoff losses (mm)
0	0.0	0.0
2	0.1	0.1
12	0.6	0.3
23	1.1	0.4
37	1.8	0.8
52	2.5	1.1
66	3.2	1.5
90	4.3	1.7
112	5.4	2.2

LDR
Initial losses:

time needed to satisfy initial losses: 9.8 min.

rain volume needed to satisfy initial losses: 0.5 mm

Variable losses:

time since start of runoff (min)	rain vol. since start of runoff (mm)	variable runoff losses (mm)
0	0.0	0
9	0.5	0.3
18	0.9	0.5
21	1.1	0.6
26	1.3	0.8
38	1.9	0.9
47	2.4	1.1
55	2.8	1.2
72	3.7	1.7
90	4.6	2.1
110	5.6	2.2

LCS
Initial losses:

time needed to satisfy initial losses: 5.8 min.

rain volume needed to satisfy initial losses: 0.3 mm

Variable losses:

time since start of runoff (min)	rain vol. since start of runoff (mm)	variable runoff losses (mm)
0	0.0	0.0
3	0.2	0.1
8	0.4	0.3
20	1.0	0.5
30	1.5	0.5
52	2.7	1.0
74	3.8	1.3
92	4.7	1.6
122	6.3	2.1

L(0)CS
Initial losses:

time needed to satisfy initial losses: 7.5 min.

rain volume needed to satisfy initial losses: 0.4 mm

Variable losses:

time since start of runoff (min)	rain vol. since start of runoff (mm)	variable runoff losses (mm)
0	0.0	0.0
2	0.1	0.1
7	0.4	0.2
17	0.9	0.5
29	1.6	0.8
55	2.9	1.3
95	5.1	2.0
127	6.8	2.8

Barker and Barker (1981) listed the requirements for an ANOVA (or factorial) calculation: the residuals must be normally distributed with a finite variance; the residuals must be independent of each other; and the residuals must be constant or homogeneous over the range of the experiment. These assumptions were therefore evaluated for the factorial models that were developed during this research.

The effects of the experimental conditions on the outcomes of the experiment were calculated by examining the change in runoff volume as the experimental conditions (such as rain intensity) were changed from low to high values. The experiments were designed using a table of contrast coefficients. The following is an example of a simple 2^2 table of contrast coefficients and some hypothetical responses:

	parameters and interactions			hypothetical
tests	A	B	AB	responses
1	-	-	+	60
2	+	-	-	72
3	-	+	-	54
4	+	+	+	68

The effect of the first experimental variable (A) is calculated by adding the responses for each test using the signs in the table under the first variable, and dividing the sum by the number of pluses in the column:

$$(-60 + 72 - 54 + 68)/2 = 13$$

while the mean is calculated:

$$(60 + 72 + 54 + 68)/4 = 63.5$$

Therefore, the responses of the first variable alone are 63.5 - 6.5 = 57 for the low value of variable A, and 63.5 + 6.5 = 70 for the high value of variable A. The effects of the second experimental variable (B) on the mean is calculated in a similar manner, noting the different signs in the table of contrast coefficients:

$$(-60 - 72 + 54 + 68)/2 = -5$$

with resulting effects on the mean of 63.5 - (-2.5) = 66 for the low value for parameter B and 63.5 + (-2.5) = 61 for the high value. An important feature of factorial experiments (especially compared to "holding everything constant except for changing one variable at a time") is the ability to examine the interaction between experimental variables. The interaction effect of parameters A and B in the above example is:

$$(60 - 72 - 54 + 68)/2 = 1$$

which results in a much smaller effect on the mean (63 and 64 for the extreme interaction conditions) than the individual parameters.

The significance of individual effects are determined by comparing them to standard error values. If the effects are less than the standard error values, then they may occur within the Gaussian distribution of the expected errors. If the effects are greater than the standard errors, then they may be significant. In the above example, if the standard error was 3, then the individual effects of parameters A and B may be significant, while the interaction effect (AB) is not significant. If the standard error is 15, then none of the parameters will be significant, and the mean value is used alone.

If replicate tests are conducted, pooled standard error values can be easily calculated directly. If no replicates are obtained, then standard errors may be estimated assuming that the higher order interactions (especially three way and greater interactions) have negligible effects and could be used as measures of errors (Box et al. 1978). Finally, normal probability plots can be drawn of all the ranked effects to identify "outliers" that are associated with possible significant experimental parameters.

The final model is then composed of the mean value, along with the significant parameter effects. If the standard error was 3 for the above example, the resulting model would therefore be:

$$Y = 63.5 + 6.5\ (A) - 2.5\ (B)$$

and the four possible outcomes compared to the measured responses (and the residuals) are as follows:

	calculated response	observed response	residuals (calc-obs)
A+B+	67.5	68	-0.5
A+B-	72.5	72	0.5
A-B+	54.5	54	0.5
A-B-	59.5	60	-0.5

The calculated residuals must also be examined to insure compliance with the assumptions: they must be normally distributed, independent of each other, and constant over the range of the experiment.

The two-level factorial experiments (only using a high and low value for each variable) described above and conducted during this research produced models assuming straight line relationships of the effects of the significant variables over their ranges. A more important use of these experiments (besides developing quantitative models) was to simply identify the most significant experimental variables that are worthy of further study. Box et al. (1978) presented many fractional factorial experimental designs that allow many variables and their lower level interactions to be examined with a relatively small number of carefully

designed experiments. When the important variables are identified, additional experiments can be conducted to interface with the initial fractional factorial experiments to examine the "shapes" of the experimental responses between the two extreme levels.

The factorial hydrology experiments conducted during this research were used to identify the significant variables affecting sheetflows across impervious surfaces. These experiments directed later analyses of impervious area runoff data obtained independently from large paved areas (Section 6) and the investigations of runoff losses for use in general runoff models (later in this section and in Section 7).

Table 5.3 is an example of the factorial calculations used during these runoff experiments. Because of the unanticipated low street loadings for the LDS experiment, the analysis was divided into three parts and was only able to examine first and second order variable interactions. Each separate factorial calculation examined two factors and their interaction. The first calculation shown on Table 5.3 considers intensity (I), street dirt loading (C), and their interaction (IC). The plus and minus signs in the columns relate to the variable values; the pluses for

Table 5.3 Example Nested Designs for 2^2 Factorial Sheetflow Hydrology Experiments

Test: Variable runoff losses (mm) after ten minutes of rain since start of runoff.

Runs Values	I	C	IC	Observed Values	S^2	Degrees of Freedom	Average
LCR, LCS, L(D)CS	-	-	+	0.20, 0.24, 0.25	0.007	2	0.23
HCR, HCS	+	-	-	0.69, 0.88	0.018	1	0.79
LDR	-	+	-	0.24	—	0	0.24
HDR, HDS	+	+	+	0.61, 0.88	0.036	1	0.75
effects:	0.54	- 0.015	-0.025	Ave = 0.50		total df = 4	

Calculation for pooled S^2 (N - 7 for replicates):

$$\text{pooled } S^2 = \frac{\Sigma(df \times S^2)}{\Sigma\, df} = \frac{0.0554}{4} = 0.0139$$

Standard error:

$$SE = (V)^{0.5}$$
$$\text{where } V\text{ (effect)} = \frac{40^2}{N} = \frac{4}{N} S^2 = \frac{4}{7}(0.0139)$$

$$V = 0.0079$$
$$SE = 0.089$$

average = 0.50 $\pm$ 0.045
I = 0.54 $\pm$ 0.09
C = -0.015 $\pm$ 0.09
IC = -0.025 $\pm$ 0.09

C	I -	I +
+	0.24	0.75
-	0.23	0.79

model: $\hat{Y} = 0.50 + 0.27\,(I)$
for I-: $\hat{Y} = 0.23$ mm
for I+: $\hat{Y} = 0.77$ mm

Table 5.3 Example Nested Designs for 2^2 Factorial Sheetflow Hydrology Experiments (Continued)

runs	I	T	IT	Observed Values	S^2	df	Average Values
LCS, L(D)CS	-	-	+	0.24, 0.25	0.0001	1	0.245
HCS, HDS	+	-	-	0.88, 0.88	0	1	0.88
LCR, LDR	-	+	-	0.20, 0.24	0.0008	1	0.22
HCR, HDR	+	+	+	0.69, 0.61	0.0032	1	0.65
effects:	0.53	-0.13	-0.10	ave. = 0.50	total df = 4		

SE = 0.023

Average = 0.50 $\pm$ 0.011
I = 0.53 $\pm$ 0.23
T = -0.13 $\pm$ 0.023
IT = -0.10 $\pm$ 0.023

$\hat{Y} = 0.50 \pm 0.05$ (IT)
for IT: $\hat{Y} = 0.45$ mm
for IT+: $\hat{Y} = 0.55$ mm

runs	C	T	CT	Observed Values	S^2	df	Average Values
HCS, LCS, L(D)CS	-	-	+	0.88, 0.24, 0.25	0.13	2	0.46
HDS	+	-	-	0.88	-	0	0.88
HCR, LCR	-	+	-	0.69, 0.20	0.12	1	0.45
HDR, LDR	+	+	+	0.61, 0.24	0.07	1	0.43
effects:	0.20	-0.23	-0.22	ave. = 0.50	total df = 4		

SE = 0.025

Average = 0.50 $\pm$ 0.13
C = 0.20 $\pm$ 0.25
T = -0.23 $\pm$ 0.25
CT = -0.22 $\pm$ 0.25

$\hat{Y} = 0.50$ mm

Overall model: Variable runoff losses (mm) after ten minutes of rain ince start of runoff:

$\hat{Y} = 0.23$ mm for low intensity rains
$\hat{Y} = 0.77$ mm for high intensity rains
with possible IT two-way interaction effect.

high intensity rains and dirty streets, and the minuses for low intensity rains and clean streets. The observed values (for variable runoff losses in this example) are the responses for each run in each variable category. The factor effects are calculated by adding and subtracting the average values as indicated for each column of signs under the factors, and dividing the result by the number of pluses in the column, as described above. Calculated standard errors are used to identify effects that are not normally distributed (these "outliers" are the significant effects). The only factor of significance identified in the first set of calculations is rain intensity. The average value is always used in the final linear model. The final model is the average value, plus and minus one-half of the effects for all of the significant factors. The two model results for this first example calculation are therefore 0.23 mm (0.50 - 0.54/2) for low intensity rains (I-) and 0.77 mm (0.50 + 0.54/2) for high intensity rains (I+).

The second and third parts of this sample calculation examines intensity (I) with street texture (T) and street cleanliness (C) with street texture (T) along with their two-way interactions. The second

calculation indicates a possible significant two-way interaction between intensity and texture (IT). The third calculation does not identify either cleanliness or texture (or their interaction) as significant when variable runoff losses are examined.

General Hydrology Factorial Calculations

These factorial calculation procedures were used to examine various responses of the controlled paved area hydrology tests. Table 5.4 presents the basic hydrology observations presented for factorial calculations. Runoff volumes were examined for various rain volume totals (only the high intensity tests had observations for rains greater than 6 mm). Runoff volume observations were also examined as functions of time of rain. Table 5.5 summarizes the factorial calculation results for these data. When examining runoff volume against rain volume, only street texture had a significant effect on the average values. For the smallest rain volume examined (1 mm), a two factor interaction between texture and intensity may also be significant. The effect of texture was much greater than this two factor interaction, however. When examining runoff volume for different rain time

Table 5.4 Values for Factorial Sheetflow Basic Hydrology Experiments

1) Observed Runoff Volume (mm) for Total Elapsed Rain:

Rain (mm)	HCR	HDR	HCS	HDS	LCR	LDR	LCS	L(D)CS
1	0.0	0.0	0.3	0.3	0.3	0.2	0.4	0.3
3	1.0	1.3	1.6	1.6	1.5	1.2	1.7	1.5
6	2.7	3.1	3.4	3.4	3.3	3.3	3.8	3.3
10	5.3	5.8	6.4	6.4	-	-	-	-
15	9.0	9.3	10.4	10.4	-	-	-	-
20	13.3	13.2	14.6	14.6	-	-	-	-

2) Observed Runoff Volume (mm) for Elapsed Time Since Rain Start:

Time (min)	HCR	HDR	HCS	HDS	LCR	LDR	LCS	L(D)CS
10	0.4	0.6	0.9	0.9	0.0	0.0	0.15	0.05
30	2.4	3.1	3.5	3.5	0.5	0.4	0.8	0.6
60	6.0	7.3	7.8	7.8	1.7	1.2	1.8	1.6
90	10.3	11.9	12.8	13.0	2.3	2.4	2.9	2.6
120	15.3	16.5	18.0	18.2	3.2	3.5	4.0	3.6

Table 5.5 Summarized Results for Factorial Sheetflow Basic Hydrology Experiments

1) Observed Runoff Volume (mm) for Total Elapsed Rain:

Rain (mm)	Runoff (mm) for: Rough Texture	Smooth Texture	Standard Error
1	0.13*	0.33*	0.04
3	1.3	1.6	0.13
6	3.1	3.4	0.16
10	5.6	6.4	(<0.8 est)
15	9.2	10.4	(<1.3 est)
20	13.2	14.6	(<1.4 est)

*Possible 2 factor interaction, intensity with texture, with 1 mm rain tests. Intensity effect much weaker than texture effect.

2) Observed Runoff Volume (mm) for Elapsed Time Since Rain Start:

Time (min)	Runoff (mm) for: Low Int.	High Int.	Standard Error
10	0.04*	0.72*	0.10
30	0.6	3.2	0.25
60	1.5	7.3	0.45
90	2.5	12.1	1.0
120	3.6	17.0	0.65

* Significant pavement texture effect for runoff after 10 minutes of rain and weaker pavement texture effects for the other time periods.

increments, very strong intensity factors were most significant. Some pavement texture interaction effects with the rain intensity were also evident, especially for the shortest rain period.

<u>Runoff Loss Factorial Calculations</u>

Table 5.6 summarizes the initial runoff losses along with the variable runoff losses for rain volume time period increments, organized for factorial calculations. Again, variable runoff losses were available only for rain volumes up to 6 mm for the low intensity tests.

Table 5.7 summarizes the factorial calculation results for this initial and variable runoff loss data. Initial losses were affected by the interaction of intensity and texture, with the texture component of the interaction appearing to be most significant. The texture effect was by far the most significant for all tests, except for the LCR and LDR tests where the low rain intensities appeared to decrease the initial losses more than expected. Further discussions of pavement texture effects on initial runoff losses presented later in this section further illustrates the unusual behavior of these low intensity tests on rough

Table 5.6 Values for Factorial Sheetflow Hydrology Experiments Examining Runoff Losses

1) Observed Initial Losses (mm):

HCR	HDR	HCS	HDS	LCR	LDR	LCS	L(D)CS
0.92	1.22	0.40	0.40	0.39	0.51	0.26	0.38

2) Variable Losses (mm) for Rain Since Runoff Start:

Rain (mm)	HCR	HDR	HCS	HDS	LCR	LDR	LCS	L(D)CS
1	0.37	0.25	0.43	0.43	0.42	0.48	0.37	0.46
3	1.12	0.86	1.19	1.19	1.25	1.36	1.08	1.35
6	2.17	1.73	2.01	2.01	2.38	2.43	1.93	2.40
10	3.60	3.08	3.18	3.18	-	-	-	-
15	5.08	4.48	4.24	4.24	-	-	-	-
20	6.17	5.80	5.02	5.02	-	-	-	-

3) Variable Losses (mm) for Time Since Runoff Start:

Time (min)	HCR	HDR	HCS	HDS	LCR	LDR	LCS	L(D)CS
10	0.69	0.61	0.88	0.88	0.20	0.24	0.24	0.25
30	2.03	1.83	2.04	2.04	0.64	0.75	0.65	0.76
60	3.87	3.70	3.63	3.63	1.26	1.41	1.19	1.45
90	5.42	5.36	4.73	4.73	1.85	1.98	1.62	2.04
120	6.46	6.83	5.48	5.48	2.35	2.43	1.99	2.56

Table 5.7 Summarized Results for 2^2 Factorial Sheetflow Hydrology Experiments Examining Runoff Losses

1) Observed Initial Runoff Losses (mm)

IT = 0.27 ± 0.09 strong intensity and <u>texture</u> interaction, especially for LCR and LDR

$\hat{Y} = 0.56 + 0.14\ (IT)$

$\hat{Y} = 0.42$ mm for IT −
$\hat{Y} = 0.70$ mm for IT +

2) Variable Runoff Losses for Rain Since Runoff Start:

Rain (mm)	Losses (mm)	Standard Error
1	0.40*	0.03
3	1.18*	0.16
6	2.13*	0.20
10	3.26	-
15	4.51	-
20	5.50	-

* Possible significant intensity effect for 1 to 6 mm rains.

3) Variable Runoff Losses for Time Since Runoff Start:

Time (min)	Losses (mm) for: Low Int.	High Int.	Standard Error
10	0.23	0.77	0.05
30	0.70	1.98	0.07
60	1.33	3.71	0.08
90	1.86	5.06	0.2
120	2.34	6.06	0.3

pavement. With either low intensity and rough streets, or high intensity and smooth streets, the initial runoff losses were 0.4 mm. The initial losses for the opposite conditions were calculated to be 0.7 mm. Initial runoff losses are expected to be most influenced by pavement texture; more detailed plots of these relationships are presented later.

The factorial calculation results summarized on Table 5.7 show that runoff variable losses were most directly related to rain volume, with no significant effects from rain intensity, street texture, street dirt loadings, or any two-way interactions of these factors. When variable runoff losses were examined against time of rain (as in Horton's infiltration equation, which will be discussed in detail in Section 7), rain intensity effects were very significant. Figure 5.3a illustrates this time dependent data. Two distinct trends are shown on this figure; one for high intensity rains and the other for low intensity rains. These two trends generally corresponded to equal rain volumes, however, as shown on Figure 5.3b. The amount of rain corresponding to a specific variable loss was about the same for both low and high intensity tests, but the rain obviously occurred over different time

Figure 5.3a Variable Runoff Losses by Time

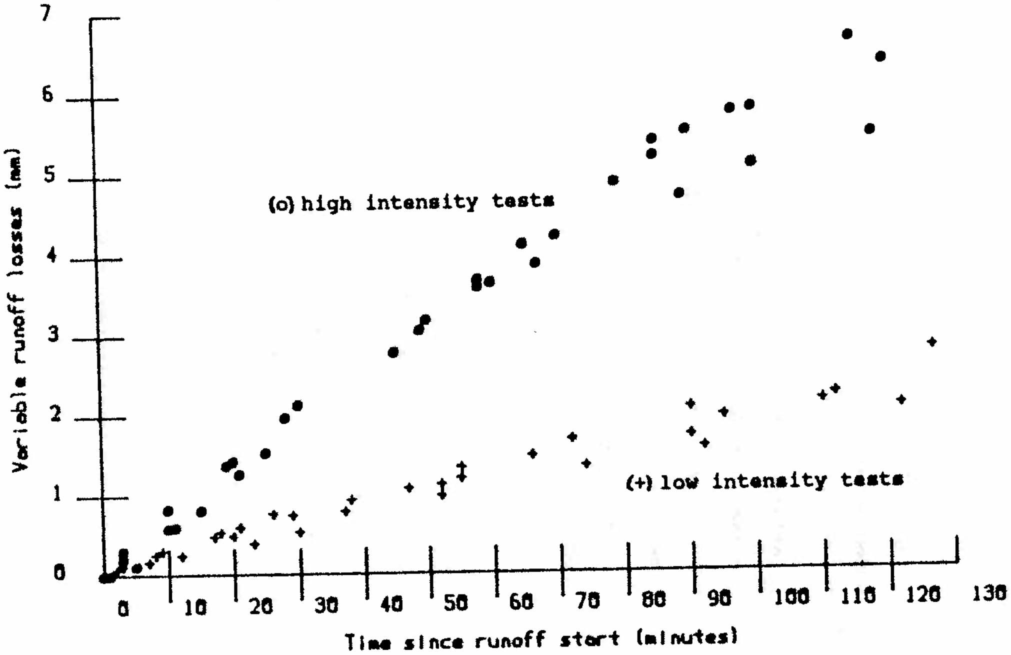
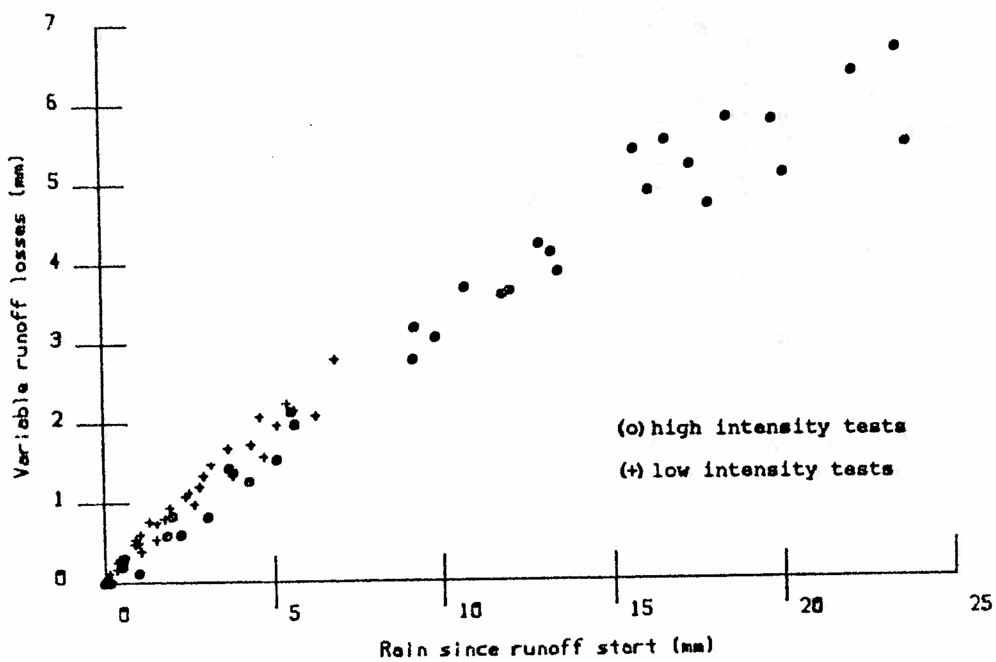


Figure 5.3b Variable Runoff Losses by Rain Depth



periods. As an example, for a variable rain loss of 2 mm, high intensity rains of 11.8 mm/hr satisfied this loss after about 30 minutes (5.9 mm of rain), while low intensity rains of 3.1 mm/hr required about 120 minutes (6.2 mm of rain).

This obvious dependence of infiltration on rain intensity may have occurred because of the increased kinetic energy associated with higher intensity rains (Springer 1976). Kinnell (1981) separated raindrop kinetic energy into two categories; the rate of expenditure of energy per unit time and the amount of rainfall kinetic energy expended per unit depth of rain. These are directly related by rain intensity. The transfer of this rain energy to the ground surface can create very large pressures over small areas. This pressure buildup is much greater for "hard" surfaces (such as for impervious surfaces), than for softer pervious surfaces. Huang et al. (1983) found that the pressures are very large, but diminish rapidly. Higher pressures also occur near the edge of the contact area. It is hypothesized that these large pressures increase infiltration of the water into the pavement. The pressure increases are not as great for the more elastic pervious surfaces and this effect may not be as

significant. However, Horton initially proposed that his infiltration model coefficients be partially determined based on rain intensity (Skaggs et al. 1969). However, the Horton infiltration parameters are not usually calibrated for different rain intensities, possibly because of reduced rain intensity influences on infiltration on pervious areas.

Runoff Polynomial Regression Equations

Second-order polynomial regression equations were fitted to the basic rainfall-runoff data for these street sheetflow tests. These data were separated by pavement texture and only rain was considered as an independent parameter in these models, based on the previous factorial calculations. Table 5.8 summarizes the SYSTAT linear regression analyses of these data. In all cases, the constants and both rain parameters were very significant and the multiple R^2 values were very close to 1.0. Even these very "good" models must be used with caution however because of the behavior of polynomials, especially beyond the limits of the data used to develop the models. As an example, the overall equation shown on Table 5.8 would predict decreasing variable runoff losses after 41.2 mm of rain, and

Table 5.8 Pavement Sheetflow Basic Hydrology Experiments

1) Smooth Textured Streets:

$$runoff = -0.27 + 0.614(rain) + 0.006(rain)^2$$

rain: 0.26 to 24.04 mm
28 observations
multiple R^2 = 0.999

Variable	Coef.	Standard Error	P(2-tail)
constant	-0.27	0.045	<0.001
rain	0.614	0.014	<0.001
$(rain)^2$	0.006	0.001	<0.001

2) Rough Textured Streets:

$$runoff = -0.414 + 0.588(rain) + 0.005(rain)^2$$

rain: 0.39 to 24.6 mm
46 observations
multiple R^2 = 0.999

Variable	Coef.	Standard Error	P(2-tail)
constant	-0.414	0.039	<0.001
rain	0.588	0.011	<0.001
$(rain)^2$	0.005	<0.001	<0.001

3) Combined Test Results:

$$runoff = -0.333 + 0.588(rain) + 0.005(rain)^2$$

rain: 0.26 to 24.6 mm
74 observations
multiple R^2 = 0.995

Variable	Coef.	Standard Error	P(2-tail)
constant	-0.333	0.073	<0.001
rain	0.588	0.021	<0.001
$(rain)^2$	0.005	0.001	<0.001

runoff volumes actually greater than the rain volume after about 83 mm of rain.

These equations could be used to calculate an initial runoff loss by identifying the rain volume when runoff is zero. The calculated initial losses were 0.44 mm for smooth pavement and 0.70 mm for rough pavement. These values compared very favorably to the initial losses calculated using the factorial procedure (0.42 and 0.70 mm for smooth and rough pavement respectively).

Simple theoretically based models fitted to actual data should be attempted if a satisfactory knowledge of the processes involved are known. However, cases of theoretically based models performing poorly when compared to "black-box" models are common and are described in Appendix C. Black-box regression models may perform well for specific sites where extensive data is available, if the equations are not extrapolated far beyond the data limits, and if the equations are reviewed for rational performance. Section 7 describes the development of theoretical models describing the runoff processes studied here. These models are compared to actual data in Section 8.

Variable Runoff Loss Polynomial Regression Equations

Second-order polynomial equations were fitted to the variable runoff loss data from the controlled pavement runoff tests, as summarized on Table 5.9. The variable parameters were all very significant and the multiple R^2 values were all close to 1.0, signifying good curve fits. However, as stated previously, good curve fits did not necessarily imply good models. Maximum variable runoff losses (and the rain volume after which these maximum variable runoff losses occurred) were calculated from these equations, and are shown in the following list:

	Maximum variable loss	Rain after which max. loss occurs
Smooth pavement	6.0	32.2
Rough pavement	8.2	41.2

These rain volumes are beyond the range of the available data and are therefore suspect because they were based solely on the regression equations.

Table 5.9 Observed Variable Runoff Losses (after initial losses are satisfied) for Pavement Sheetflow Experiments

1) Smooth Textured Streets:

losvar = $-0.17 + 0.386(\text{rain}) - 0.006(\text{rain})^2$

rain (total rain in event): 0.26 to 24.04 mm
28 observations
multiple R^2 = 0.994

Variable	Coef.	Standard Error	P(2-tail)
constant	-0.17	0.045	<0.001
rain	0.386	0.014	<0.001
$(\text{rain})^2$	-0.006	0.001	<0.001

2) Rough Textured Streets:

losvar = $-0.286 + 0.412(\text{rain}) - 0.005(\text{rain})^2$

rain (total rain in event): 0.39 to 24.6 mm
46 observations
multiple R^2 = 0.996

Variable	Coef.	Standard Error	P(2-tail)
constant	-0.286	0.039	<0.001
rain	0.412	0.011	<0.001
$(\text{rain})^2$	-0.005	<0.001	<0.001

3) Combined Test Results:

losvar = $-0.227 + 0.412(\text{rain}) - 0.005(\text{rain})^2$

rain (total rain in event): 0.26 to 24.6 mm
74 observations
multiple R^2 = 0.974

Variable	Coef.	Standard Error	P(2-tail)
constant	-0.227	0.073	0.003
rain	0.412	0.021	<0.001
$(\text{rain})^2$	-0.005	0.001	<0.001

D. Relationships of Initial Runoff Losses with Measured Pavement Texture

Each of the pavement test areas were selected to provide the range of conditions needed for the factorial experimental design requirements. The texture of the pavement at each site was directly measured from plaster casts (about 15 cm across). Latex positives were also made of each plaster negative. Micro-scale topography was measured from the plaster casts by using a wire profile step-gauge, having a horizontal resolution equal to the wire diameters (about 1 mm). Profiles were obtained with the step-gauge at six locations on each cast. The step-gauge was photographed for each profile transect and enlarged for measurement. Cross-sectional areas of potential pools were measured using a planimeter for five different pavement slopes (0 to 10 percent slopes).

Table 5.10 summarizes the surface roughness characteristics. Actual water accumulations before runoff from the latex positives were also directly measured and were found to closely agree with these cross-sectional measurements. Detention storage, the

Table 5.10 Surface Roughness Characteristics

	Street Slope (percent): 0	1	2	5	10%
Detention Storage (mm):					
rough streets: average	1.28	1.19	1.02	0.72	0.45
: range	0.95-1.64	0.92-1.49	0.71-1.41	0.43-0.98	0.24-0.69
smooth streets: average	0.62	0.51	0.40	0.21	0.11
: range	0.31-0.85	0.28-0.80	0.20-0.65	0.083-0.37	0.036-0.26
Surface Ponding (%):					
rough streets: average	100%	96%	89%	78%	66%
: range	100	91-99	81-95	72-85	53-74
smooth streets: average	100%	93%	84%	58%	38%
: range	100	76-99	71-94	41-87	9-56
Average Pond Depths (mm):					
rough streets: average	1.28	1.24	1.15	0.92	0.68
: range	0.95-1.64	0.97-1.59	0.79-1.52	0.57-1.31	0.37-1.11
smooth streets: average	0.62	0.54	0.48	0.37	0.29
: range	0.31-0.85	0.34-0.82	0.24-0.69	0.17-0.57	0.072-0.54

ponding volume per pavement surface area (reported as a depth), was measured from the profiles described above. The rough pavement had significantly greater detention storage volumes than the smooth pavement for all slope conditions. As the pavement slopes increased, the detention storages decreased. At 10 percent slopes, the detention storage was only about 1/6 to 1/3 of the level street detention storage volumes. The smooth streets experienced a much greater percentage decrease in detention storage with slope than the rough streets.

By definition, level pavement was assumed to be ponded over its entire surface. Pond depths were also at their maximum for level conditions and were about twice as deep for the rough pavement as for the smooth pavement. As the slope increased, the surface area that was ponded and the average pond depths decreased.

These directly measured detention storage volumes were compared to the initial runoff storage losses determined from the pavement sheetflow tests. Table 5.11 shows the measured detention storage volumes and the differences between these measured detention storage volumes and the initial runoff losses for each runoff test. Factorial calculation results for these data are summarized on Table 5.12. For measured

Table 5.11 Values for Factorial Tests Investigating Texture Related Detention/Storage and Initial Runoff Losses

1) Measured Street Texture Related Detention/Storage (mm):

HCR	HDR	HCS	HDS	LCR	LDR	LCS	L(D)CS
1.05	1.05	0.42	0.42	1.05	1.05	0.29	0.29

2) Difference of Measured Street Texture Minus Observed Initial Losses (mm):

HCR	HDR	HCS	HDS	LCR	LDR	LCS	L(D)CS
0.13	-0.17	0.02	0.02	0.65	0.54	0.03	-0.09

Table 5.12 Summarized Results for Factorial Tests Investigating Texture Related Detention/Storage and Initial Runoff Losses

1) Measured Street Texture (mm):

$T = 0.68 \pm 0.04$

$\hat{Y} = 0.7 + 0.34\ T$

$\hat{Y} = 0.36$ mm for smooth streets
$\hat{Y} = 1.04$ mm for rough streets

2) Measured Street Texture Minus Observed Initial Runoff Losses (mm):

$IT = 0.34 \pm 0.085$ Strong intensity and <u>texture</u> interaction, especially for LCR and LDR

$\hat{Y} = 0.14 + 0.17\ IT$

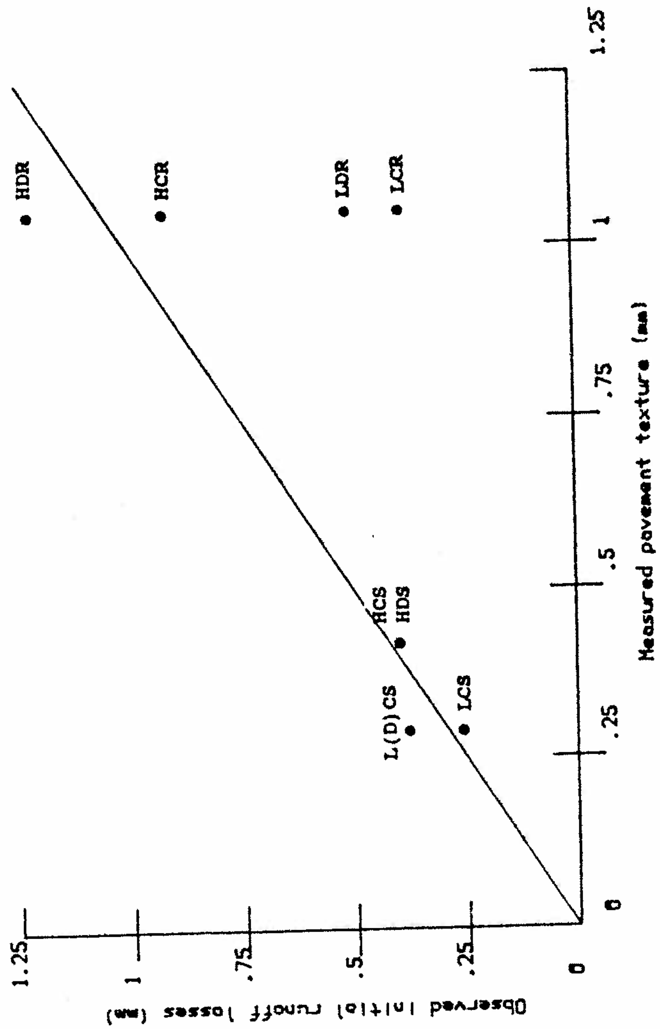
$\hat{Y} = -0.03$ for IT−
$\hat{Y} = 0.31$ for IT+

detention storage, street texture alone was significant (rain intensity and dirt loadings would not affect the measured detention storage volumes). The differences between detention storage and initial runoff losses, however, were dependent on the interaction between texture and rain intensity, with texture being most significant for most cases.

Figure 5.4 plots the observed initial runoff losses against the measured pavement detention storage values. The observed initial runoff losses for the LDR and LCR tests were much lower than expected, compared to the favorable relationships between initial losses and detention storage values observed for the other tests. The low rain intensity tests on rough pavement showed initial losses of about one-half of the measured detention storage values.

The previous discussion on variable losses (Figure 5.3) showed that high and low intensity rains experienced similar infiltration losses for the same rain depths. However, the high intensity rains experienced these losses over a shorter period than the low intensity rains. Increased infiltration rates during high intensity rains may have "emptied" the surface detention/storage volumes, allowing subsequent

Figure 5.4 Plot of Initial Runoff Losses and Pavement Texture Detention Storage Volume



"refilling" and therefore greater detention/storage losses. The low intensity rough street tests may also have had lower initial runoff losses because of more efficient runoff, for unknown reasons. The high intensity tests appeared to better represent the anticipated initial runoff losses for rough streets because of their better relationship with the detention/storage measurements. The low intensity tests had less initial runoff losses than expected.

E. Other Initial and Variable Runoff Losses

Sorption of water by street dirt was another initial runoff loss mechanism investigated during this research. Samples of street dirt collected in the test areas were found to be extremely hydrophobic, possibly because of typical oil and grease contamination. Drops of water were found to roll off street dirt with very little sorption. In fact, the rolling drops carried some dirt on their outside as they rolled across the street dirt. Maximum sorption occurred on the 2000 to 6400 micron sized small pebbles, but was still less than 0.1 ml of water per gram of street dirt. Water sorption tests were also conducted on dried garden soil (clay-loam) for comparison. The garden soils readily sorbed water at a rate of about 0.7 ml of water per gram of soil, or at least ten times greater than the street dirt. The maximum potential street dirt loading at the Toronto test sites was about 300 grams per curb-mile, or about 60 grams of dirt per square meter of street. Maximum water sorption associated with initial runoff losses would therefore be about 0.004 mm, which is very small when compared with the detention storage losses. If the street dirt behaved like garden soil, then the maximum water sorption losses would still only be about 0.05 mm. Sorption of water by street dirt can therefore be considered an insignificant initial runoff loss.

Flash evaporation of rain falling on hot pavement was also investigated as a possible important initial runoff loss mechanism. Simple calculations using the heat of vaporization of water (1050 BTU/lb water), the heat capacity of pavement (about 20 BTU/cubic foot - ^{o}F for concrete), a 30^{o}F temperature difference between the water and pavement, and a pavement thickness of 3 inches showed that about 1/4 inch (about 6.5 mm) of rain may be evaporated before the pavement cools 30^{o}F.

About 1 inch (25 mm) of rain per hour could be evaporated (up to a total of about 1/4 inch, or 6.5 mm), assuming a thermal conductivity of about 7.5 BTU per hr per ft^2 per ^{o}F per inch for concrete. Actual flash evaporation is probably much less (probably only about 1 to 10 percent of 6.5 mm) because most of the water is continuously flowing across the pavement and along the gutter and is not heated quickly. Typical pavement contact times for sheetflows across streets is about 10 seconds, with maybe another few minutes of flow along the gutter before entering the storm drainage system.

Grimmond, et al (1986) found a total of only about 0.3 mm of rain evaporation in a test watershed in Vancouver. This was equal to about 3 percent of the total rain from a typical 3 hr, 10 mm rain. Evaporation is therefore also a relatively insignificant runoff loss mechanism for most cases. Evaporation of pooled water, especially on flat roofs or on rough pavement or unpaved parking areas, can be very important. Besides infiltration, evaporation is the only mechanism capable of emptying detention storage. If the detention storage is emptied during the rain, increased runoff losses would occur.

F. Summary of Small Paved Area Hydrology Tests

This section summarizes the rainfall-runoff relationships and runoff losses observed during controlled pavement sheetflow tests. The tests examined runoff volumes, initial losses, and variable losses for different pavement textures, rain intensities, rain periods, rain volumes, and street dirt loadings.

Factorial calculations identified pavement texture as the most significant factor influencing runoff volume for any specific rain volume, and rain intensity as the most significant factor for any specific rain duration. This rain intensity factor for specific rain durations reduced directly to rain volume (duration times intensity) as the most important factor influencing runoff volume: rain intensity by itself was not a significant factor in determining runoff volume. Second-order polynomial regression equations (only using rain as the independent parameters) fit the runoff data very well, but behaved poorly beyond the range of observed data.

Initial runoff losses were mostly influenced by pavement texture, but a confusing (and unexplained) interaction of texture and rain intensity was also significant for low intensity rains on rough pavement. Pavement roughness was directly measured to calculate detention storage for different pavement slopes and textures. These detention storage measurements agreed quite well with the monitored initial runoff losses (with the exception noted above). Sorption of water by street dirt was also examined as a potential initial runoff loss mechanism, but was found to be insignificant because of the hydrophobic character of street dirt, possibly due to oil and grease contamination.

Variable runoff losses (infiltration) were influenced by rain volume (intensity times duration), and not by intensity or duration alone. Section 7 examines pavement infiltration in more detail, comparing the hypothesized runoff (and loss) model to the Horton infiltration equation and the SCS curve number method.

Flash and continuous evaporation were examined as potential processes affecting initial and variable runoff losses, respectively. Flash evaporation is expected to have minimal influence on initial losses, but continuous evaporation may be important when considering rough paved surfaces, unpaved parking areas, and especially flat roofs.

These controlled tests have demonstrated the important roles that pavement texture had in determining initial runoff losses and that rain volume had in determining variable runoff losses (infiltration). Runoff volume was mostly dependent on rain volume, but pavement texture was also important, especially for small events where initial losses were large portions of the total runoff losses. Rain intensity had very little effect on either initial or variable runoff losses, or on runoff volume.

Section 6 examines large paved parking lot and large flat roof runoff data obtained during actual rains to examine the general applicability of the small-scale test area results reported in this section.

SECTION 6

LARGE-SCALE PAVED AREA HYDROLOGY OBSERVATIONS

A. Introduction

The Nationwide Urban Runoff Program (NURP) project conducted in Milwaukee County (Bannerman et al. 1983) included extensive monitoring of a shopping center. There were two monitoring locations at the shopping center, one sampled runoff from a paved parking area, while another sampled runoff from the rest of the parking area and a large flat roof. This section examines the basic hydrology data obtained at these two locations. These data enabled the paved area runoff models, identified in the previous section, to be examined for transferability. The Milwaukee data were collected from large impervious areas, during three years, and included a number of very large rains, therefore allowing the applicability of the small-scale test results presented in the previous section to be extended for a broader range of conditions. The next section uses these data, in conjunction with the small-scale paved area test results, to develop a general paved area runoff model.

B. Milwaukee NURP Shopping Center Site Description and Runoff Data

Table 6.1 describes the two shopping center monitoring areas. Both areas were about five hectares in size, but one was mostly a paved parking area ("Post Office") while the other was about evenly divided between a paved parking area and a flat roof area ("Rustler"). Tables 6.2 and 6.3 list the basic hydrology data for these two sites, separated by season. About 75 rains were monitored at each station; most of the observations were obtained during the summer and spring months. Most of the rains were less than 25 mm, while two very large rains monitored at each location were greater than 70 mm, making them among the largest urban runoff events ever monitored for both hydrology and water quality data.

Table 6.1 Milwaukee NURP Shopping Center Site Characteristics

1) Land Covers:	"Post Office"	"Rustler"
Streets	11%	10%
Paved parking	86	48
Connected roofs	<1	42
Other	3	<1
Total:	100%	
Area:	4.8 ha	5.0 ha

2) Land Development Characteristics:

- Roofs: All drains directly connected to storm sewerage and are flat tar and gravel.
- No nearby sediment sources or treated wood
- Very little landscaping (some grass strips)
- Flat topography (<5%)
- Clean to fair litter loadings
- Light to moderate traffic densities and slow speeds (<25 mph)
- Parking and pavement condition:

"Post Office" % of parking area	Parking density	Pavement condition	Pavement texture	"Rustler" % of parking area	Parking density	Pavement condition	Pavement texture
25%	light	fair to poor	inter.	25%	moderate	fair to poor	inter. to rough
35%	light	fair	inter.	75%	light	fair	inter.
40%	heavy	fair	smooth to inter.				

Table 6.2 Milwaukee NURP "Post Office" Rainfall-Runoff Observations

Spring date	Spring rain (mm)	Spring runoff (mm)	Summer date	Summer rain (mm)	Summer runoff (mm)	Fall date	Fall rain (mm)	Fall runoff (mm)
4/3/80	5.84	5.59	6/1/80	7.37	7.09	10/2/80	0.51	0.28
4/6	2.54	1.88	6/2	6.35	6.43	10/16	3.81	3.61
4/8	10.92	10.21	6/5	22.61	21.06	10/16	13.46	12.90
4/9	8.13	7.93	6/6	18.29	15.88	10/24	5.08	4.85
4/14	4.83	4.67	6/7	15.24	11.02	11/13	13.97	13.23
4/28	2.54	1.60	6/19	2.29	1.65	11/23	4.32	4.12
4/28	20.32	19.84	7/16	26.42	25.43	12/1	2.03	1.47
5/13	5.33	4.22	8/2	6.60	6.07	12/5	16.26	15.47
4/4/81	22.10	21.18	8/2	6.35	4.93	10/14/81	26.92	25.96
4/8	8.13	7.49	8/4	70.87	60.66	10/17	20.57	19.69
4/8	9.40	9.17	8/7	24.13	22.99			
4/10	42.16	36.25	8/11	14.48	13.61			
4/13	30.73	30.00	8/13	1.27	1.04			
5/29	4.06	2.29	8/19	14.73	14.22			
5/29	5.59	4.37	9/9	32.00	21.64			
4/2/82	7.11	5.89	9/12	34.80	33.73			
4/2	9.65	8.89	9/16	19.56	18.47			
4/2	10.92	10.49	9/20	18.29	16.99			
4/3	24.38	21.64	9/25	2.29	1.78			
5/11	12.45	11.00	6/21/81	22.61	19.79			
5/15	4.32	3.61	7/12	29.72	26.75			
5/21	12.70	12.17	7/12	16.26	13.59			
5/22	13.46	8.94	7/13	90.42	83.57			
5/26	5.59	5.18	7/17	14.99	11.81			
5/27	5.08	3.56	7/20	6.86	6.33			
			7/27	11.18	10.36			
			8/14	18.29	17.65			
			8/15	0.25	0.10			
			8/26	25.91	24.92			
			8/27	5.84	5.49			
			8/29	4.83	4.06			
			8/31	46.99	45.34			
			9/7	13.97	13.54			
			9/25	2.29	2.03			
			9/26	7.37	7.06			
			9/30	35.56	33.83			
			9/30	4.32	3.66			
			6/7/82	4.57	3.43			
			6/12	1.78	1.40			
			6/15	19.30	18.59			
			6/20	3.30	2.41			
number 25	25	25	41	41	41	10	10	10
minimum -	2.54	1.60	-	0.25	0.10	-	0.51	0.28
maximum -	42.16	36.25	-	90.42	83.57	-	26.92	25.96
mean -	11.53	10.32	-	17.82	16.11	-	10.69	10.16
std. dev. -	9.61	8.89	-	18.29	16.71	-	8.87	8.60

Table 6.3 Milwaukee NURP "Rustler" Rainfall-Runoff Observations

Spring date	Spring rain (mm)	Spring runoff (mm)	Summer date	Summer rain (mm)	Summer runoff (mm)	Fall date	Fall rain (mm)	Fall runoff (mm)
4/3/80	5.33	4.50	6/5/80	22.61	20.96	10/3/80	1.27	0.61
4/9	8.13	6.78	6/27	9.91	7.98	10/16	2.29	1.27
4/14	4.83	3.78	7/16	26.42	24.82	10/17	14.48	14.12
5/13	5.33	3.61	7/26	46.23	44.22	10/24	5.33	3.68
4/22/81	4.83	3.23	8/2	46.23	43.08	11/23	3.56	2.31
5/10	14.22	11.13	8/7	22.86	21.13	12/8	5.84	4.42
5/23	2.03	0.89	8/11	14.22	12.17	10/14/81	27.18	25.65
5/29	4.06	2.49	8/13	1.27	0.66	10/17	10.16	8.56
3/12/82	5.59	4.93	8/16	9.91	8.31	10/17	11.68	11.15
3/16	7.62	6.17	8/19	12.19	10.44			
3/19	8.89	7.39	9/20	18.80	17.20			
4/2	7.87	5.59	9/22	22.61	20.68			
4/2	9.91	8.15	9/22	6.60	5.36			
4/2	20.57	20.19	9/25	2.03	1.25			
4/3	19.05	16.69	6/8/81	3.30	2.11			
4/16	22.61	21.79	6/15	2.54	1.19			
5/11	9.91	8.13	6/21	21.08	17.17			
5/15	3.56	2.16	7/12	29.72	21.92			
5/21	11.18	8.74	7/12	16.00	11.13			
5/22	12.19	10.74	7/13	71.88	70.26			
5/26	5.33	4.34	8/7	2.54	1.58			
5/27	4.06	3.12	8/14	15.24	12.88			
			8/15	9.40	5.87			
			8/26	25.91	24.31			
			8/27	5.84	5.66			
			8/31	46.48	44.68			
			9/21	7.11	5.72			
			9/30	35.56	30.96			
			6/7/82	3.56	1.93			
			6/12	1.78	1.47			
			6/15	18.80	16.76			
			6/20	19.30	18.01			
			6/25	8.38	6.50			
			6/29	4.57	3.12			
number 22	22	22	34	34	34	9	9	9
minimum -	2.03	0.89	-	1.27	0.66	-	1.27	0.61
maximum -	22.61	21.79	-	71.88	70.26	-	27.18	25.65
mean -	8.96	7.47	-	17.97	15.93	-	9.09	7.98
std. dev. -	5.70	5.66	-	16.14	15.72	-	8.12	8.09

C. Basic Rainfall-Runoff Linear Regression Relationships for Milwaukee NURP Shopping Center Site

SYSTAT was used to test simple first and second order linear regression equations for the rain and runoff volume data. Tables 6.4 through 6.6 summarize the parameter coefficients and fits of these regression equations. Tables 6.4 and 6.5 show separate equations for the different seasons and for the combined time periods. Table 6.6 shows a single equation for the two sites combined and for all seasons.

When the runoff data were plotted against the rain data, no visible differences were noted for the different seasons and between the two sites. The Post Office equations showed very little difference in slopes (rain coefficients) for the different seasons, while the constant terms (although small) showed larger differences, but were not significant in many cases. Variations in the equations' parameters for the Rustler data were greater, with significant second order rain coefficients for one season and for the three seasons combined. All of the multiple R^2 values for all nine equations were very close to 1.0, indicating nearly "perfect" data fits with the regression equations that

Table 6.4 General Rainfall-Runoff Relationships for "Post Office" Milwaukee NURP Site

1. Spring Runoff:

Runoff = -0.259 + 0.918 (rain)
Rain: 2.5 to 42.2 mm
25 observations
multiple R^2 = 0.99

Variable	Coef.	Standard Error	P(2-tail)
constant	-0.259	0.354	0.472*
rain	0.918	0.024	< 0.001

2. Summer Runoff:

Runoff = -0.094 + 0.909 (rain)
rain: 0.3 to 90.4 mm
41 observations
multiple R^2 = 0.99

Variable	Coef.	Standard Error	P(2-tail)
constant	-0.094	0.370	0.802*
rain	0.909	0.015	< 0.001

3. Fall Runoff:

Runoff = -0.209 + 0.969 (rain)
rain = 0.5 to 26.9 mm
10 observations
multiple R^2 = 1.00

Variable	Coef.	Standard Error	P(2-tail)
constant	-0.209	0.073	0.021
rain	0.969	0.005	< 0.001

4. All Seasons Combined:

Runoff = -0.086 + 0.912 (rain)
rain: 0.3 to 90.4 mm
76 observations
multiple R2 = 0.99

Variable	Coef.	Standard Error	P(2-tail)
constant	-0.086	0.226	0.70*
rain	0.912	0.011	< 0.001

* These equation variables were "not significant", but were used to calculate initial runoff losses.

Table 6.5 General Rainfall-Runoff Relationships for "Rustler" Milwaukee NURP Site

1. Spring Runoff:

Runoff = -0.197 + 0.715 (rain) + 0.011 (rain)²
Rain: 2.0 to 22.6 mm
22 observations
multiple R^2 = 0.99

Variable	Coef.	Standard Error	P(2-tail)
constant	-0.197	0.452	0.669*
(rain)	0.715	0.093	< 0.001
(rain)²	0.011	0.004	0.007

2. Summer Runoff:

Runoff = -0.68 + 0.87 (rain) + 0.002 (rain)²
rain: 1.3 to 71.9 mm
34 observations
multiple R^2 = 0.99

Variable	Coef.	Standard Error	P(2-tail)
constant	-1.068	0.443	0.135*
(rain)	0.87	0.039	< 0.001

3. Fall Runoff:

Runoff = -1.063 + 0.995 (rain)
rain = 1.3 to 27.2 mm
9 observations
multiple R^2 = 1.00

Variable	Coef.	Standard Error	P(2-tail)
constant	-1.063	0.271	0.006
(rain)	0.995	0.023	< 0.001

4. All Seasons Combined:

Runoff = -0.788 + 0.898 (rain) + 0.001 (rain)²
rain: 1.3 to 71.9 mm
65 observations
multiple R^2 = 0.99

Variable	Coef.	Standard Error	P(2-tail)
constant	-0.788	0.255	0.003
(rain)	0.898	0.026	< 0.001
(rain)²	0.001	< 0.001	0.006

* These equation variables were "not significant", but were used to calculate initial runoff losses.

Table 6.6 General Rainfall-Runoff Relationships for the Two Large Parking Area Milwaukee NURP Sites Combined

Runoff = 0-.667 + 0.934 (rain)
Rain: 0.3 to 90.4 mm
131 observations
multiple R^2 = 0.99

Variable	Coef.	Standard Error	P(2-tail)
constant	-0.667	0.170	< 0.001
(rain)	0.934	0.008	< 0.001

only considered rain depth. As stated in the previous section, good fitting regression equations do not necessarily indicate good models.

D. Runoff Losses for Milwaukee NURP Shopping Center Site

Initial Runoff Losses

The rainfall-runoff regression equations were used to estimate initial runoff losses by solving for the rain when runoff was zero. The nonsignificant constant terms were therefore needed in order to calculate non-zero initial loss estimates. These initial loss estimates, shown on Table 6.7, were almost all less than 1 mm. Because of the high probabilities that the equation constant terms were zero, the initial losses were probably quite small. The initial losses for the Rustler site appear to be larger than for the Post Office site, possibly indicating greater initial runoff losses for flat commercial roofs than for large paved parking areas.

Table 6.7 Initial Runoff Losses for Two Large Parking Area Milwaukee NURP Sites

- Calculated from rainfall-runoff equations shown on Tables 6.4 through 6.6 when setting runoff = 0.0.

	Inital Losses (mm)	(inch)
1. "Post Office" site		
- spring	0.28	0.011
- summer	0.10	0.004
- fall	0.22	0.009
- all seasons combined	0.09	0.004
2. "Rustler" site		
- spring	0.27	0.011
- summer	0.78	0.031
- fall	1.07	0.040
- all seasons combined	0.88	0.035
3. Both sites combined for all seasons	0.71	0.028

Variable Runoff Losses

Variable runoff losses were determined for each observation in Tables 6.1 and 6.2 by subtracting both runoff volumes and initial runoff losses (from Table 6.7) from the rain volumes. SYSTAT was used to determine the model multiple R^2 values, along with the parameter coefficients, standard errors, and the two-tailed probabilities of significance for each simple linear regression equation for these adjusted data. Tables 6.8 through 6.10 summarize these equations. These equations only include a slope term, or the direct ratio between variable runoff losses (losvar) and rain. Constant terms were examined, but were far from significant and their presence did not appreciably affect the rain coefficient values. These equation parameter coefficients were all very significant, with one exception; fall season for Rustler. However, the multiple R^2 values were all quite poor, indicating relatively large amounts of data scatter not explained by the equations.

Multiple R^2 values can be misleading for equations having very small slopes (such as in this case). If the dependent variable is a constant (the equation having no slope term), the multiple R^2 value would be zero,

Table 6.8 Observed Variable Runoff Losses for "Post Office" Milwaukee NURP Site

1. Spring Variable Losses:

losvar = 0.081 (rain)
multiple R^2 = 0.56

Variable	Coef.	Standard Error	P(2-tail)
(rain)	0.081	0.015	< 0.001

2. Summer Variable Losses:

losvar = 0.091 (rain)
multiple R^2 = 0.66

Variable	Coef.	Standard Error	P(2-tail)
(rain)	0.091	0.010	< 0.001

3. Fall Variable Losses:

losvar = 0.030 (rain)
multiple R^2 = 0.91

Variable	Coef.	Standard Error	P(2-tail)
(rain)	0.030	0.003	< 0.001

4. All Seasons Combined:

losvar = 0.088 (rain)
multiple R^2 = 0.99

Variable	Coef.	Standard Error	P(2-tail)
(rain)	0.088	0.008	< 0.001

Table 6.9 **Observed Variable Runoff Losses for "Rustler" Milwaukee NURP Site**

1) Spring Variable Losses:

losvar = 0.102(rain)
multiple R^2 = 0.61

Variable	Coef.	Standard Error	P(2-tail)
(rain)	0.102	0.018	< 0.001

2. Summer Variable Losses:

losvar = 0.053 (rain)
multiple R^2 = 0.43

Variable	Coef.	Standard Error	P(2-tail)
(rain)	0.053	0.010	< 0.001

3. Fall Variable Losses:

losvar = 0.005 (rain)
multiple R^2 = < 0.02

Variable	Coef.	Standard Error	P(2-tail)
(rain)	0.005	0.014	0.73*

4. All Seasons Combined:

losvar = 0.048 (rain)
multiple R^2 = 0.40

Variable	Coef.	Standard Error	P(2-tail)
(rain)	0.048	0.007	< 0.001

* "not significant"

Table 6.10 **Observed Variable Runoff Losses for the Two Large Parking Area Milwaukee NURP Sites Combined**

losvar = 0.064 (rain)
multiple R^2 = 0.49

Variable	Coef.	Standard Error	P(2-tail)
(rain)	0.064	0.006	< 0.001

indicating no relationship with the independent parameter. When the slopes are low, the multiple R^2 values approach zero because of this apparent non-relationship, not necessarily because of poorly fitting equations. The slope terms in almost all of these equations were found to be very significant, while the constant terms were all very insignificant, actually indicating a significant relationship between variable runoff losses and rain.

E. Summary of Simple Hydrology Relationships for Milwaukee NURP Shopping Center Site

The monitoring data obtained from the Milwaukee NURP shopping center site was very unique in that it provided substantial data from large paved parking and flat roof areas collected over several years and for a wide range of rain conditions. The data was examined during this dissertation research to provide an independent data base that could be used to substantiate the theoretical urban hydrology relationships developed from the small-scale Toronto paved area tests.

This section presented this Milwaukee data and provided simple regression equations describing the observed rainfall-runoff relationships and the initial and variable runoff losses. The forms of these relationships were very similar for the small-scale and large-scale test data, indicating good transferability of these types of relationships for a broad range of pavement and rain conditions. The next section develops theoretical urban hydrology paved area relationships based on the extensive small-scale and large-scale data presented in the past three sections and the literature.

SECTION 7

GENERAL PAVED AREA HYDROLOGY MODEL AND COMPARISONS WITH THE SCS CURVE NUMBER PROCEDURE AND THE HORTON INFILTRATION EQUATION

A. Introduction

This section presents the hypothesized general model describing runoff from paved areas developed as part of this research. This general model is also compared to the Soil Conservation Service curve number procedure and the Horton infiltration equation. The previously presented pavement runoff test data are evaluated for these three runoff prediction procedures. The relationships of the different procedures to each other are also described, including how the general model can be used to determine appropriate SCS and Horton equation parameters in existing stormwater drainage design models. The next section verifies the hypothesized general paved area runoff model (in conjunction with a pervious area runoff model) by examining outfall runoff data from complex urban

watersheds that were monitored in Toronto and Milwaukee.

B. Background

Runoff Losses

Urban area hydrology components, especially for small impervious areas, are typically accepted as the most accurate portions of urban runoff models. Novotny and Chesters (1981) stated that these analyses are accurate to within a few percent. However, if there are any inaccuracies in the hydrology portions of a model, then the other related model components magnify these errors. Lazaro (1979) reported that if researchers could gain a better understanding of the hydrology of small inlet areas, then it would be a simple procedure to simulate hydrographs for larger areas.

Unfortunately, impervious area flow estimates are assumed to be much more accurate than warranted. When extensive field studies have been conducted simultaneously with modeling efforts, major differences in "actual" and modeled urban runoff parameters have been noted (Pitt and Bissonnette 1984). The common assumption that the majority of urban runoff flows always come from impervious areas is incorrect. The current prediction methods used to estimate runoff from impervious areas therefore have some problems. One of the major research topics of this dissertation was the investigation of the hydrology of impervious areas during common small storms. The results of this research have been compared to results obtained from other commonly used runoff prediction processes (SCS curve number procedure and the Horton equation).

The differences between the depths of rain that fall in an urban area and the amounts of runoff generated are the losses associated with various mechanisms. Models address these losses somewhat differently, but typically include an infiltration relationship for pervious areas (the Horton equation is common), and an empirical relationship for surface detention storage for impervious areas.

The way that different urban runoff models deal with these losses is important. The runoff models that are used in urban runoff quality studies should be distinct from the runoff models that are used to design drainage facilities because of the effects of the modeling assumptions on runoff yields for different

sized storms. Simple approximations may also be adequate for some users, while others require more complex models. The following discussion summarizes the typical processes used by urban hydrology models to estimate these runoff losses from impervious areas.

<u>General Runoff Loss Mechanisms for Impervious Surfaces</u>

When rain falls on an impervious surface, much of it will flow off the surface and contribute to the total urban runoff. Some will be lost in various ways, including:

- o interception by over-hanging vegetation before it reaches the impervious surface,
- o flash evaporation caused by the heat of the surface,
- o depression storage, where rain is captured in surface depressions or by surface tension for later evaporation or infiltration, and
- o sorption by street dirt.

These losses are mostly associated with the initial portions of the rain and are termed initial abstractions. After they become satisfied, runoff begins. Many modeling procedures and field investigations lump these losses together and assume that detention storage is the most important initial loss mechanism.

Water may also infiltrate through pavement, or through cracks or seams in the pavement. If the impervious surface is not directly connected to the drainage system, overland flow away from the paved area would be further reduced by infiltration. For small rains, a much greater portion of the rain will be lost to these processes than for large rains.

Aron (1982) concluded that runoff losses, including interception, depression storage, and infiltration, were among the most important factors in estimating runoff, but were subject to the largest amount of uncertainty. He considered the lack of understanding of these losses to be the weakest link in hydrology modeling. Even though much effort had been spent in refining urban hydrology modeling, he stated that the selection of the parameters for the loss equations usually relied on experimental data from the 1940s. Aron recognized the importance of impervious area loss effects on estimating peak flood flow rates for most urban areas. He also found that runoff volume estimates (for large events) were also significantly affected by runoff from pervious areas.

Several studies have examined total runoff losses from impervious surfaces (paved roads, paved parking areas, and different types of roofs). Roof runoff losses are assumed to include the initial abstractions, with no infiltration losses, while the surface paved area losses also include infiltration. Very few attempts have been made to decompose these losses into the component parts.

General Initial Abstractions, Stressing Depression/Storage

Davies and Hollis (1981) examined runoff from three roofs and a section of road in Hertfordshire, England. Even during cold moist days they observed significant runoff losses from these "impervious" surfaces. Inclined roofs had runoff yields of about 75 percent of the rainfall, while the flat roofs and paved areas had runoff yields equaling only about 20 percent of the rainfall. They estimated that detention storage was about 0.25 mm for the inclined roofs and about 1 mm for the road. They felt that depression storage was more important than evaporation for the roofs, while infiltration and depression storage were most important for the road site.

Brater (1968) summarized values of initial abstractions that have been used for most modeling studies. Tholin and Keifer (1960) suggested initial abstraction values of 1.6 mm for pavement and initial abstraction values of 6.4 mm for grass area. Hicks in 1944 (as reported by Aron 1982) recommended maximum initial abstraction values of 0.5 mm for sand, 2.5 mm for clay, and 4 mm for loam soils. Viessman (1966) recommended initial abstraction values ranging from 1.0 to 2.5 mm for small paved areas. Aron (1982) reported that the Denver Regional Council of Governments used initial abstraction values of 7.5 mm for lawns, 10 mm for wooded fields and open areas, 2.5 mm for large paved areas and flat roofs, and 1.3 mm for sloped roofs. Since these values were mainly used for flood analysis, these small pavement initial abstractions did not significantly affect peak flood flow rate estimates and were therefore not examined in detail. Brater, however, found that pavement initial abstractions could not be ignored when examining small storms.

Davies and Hollis (1981) found that initial abstractions could not be found by simply regressing rainfall against runoff because of the large scatter of data for small events (the resulting error values were

significantly greater than the measured values). They also concluded that initial abstractions contained other losses besides detention storage. Morel-Seytoux et al. (1982) also detected significant rainfall intensity effects on initial abstraction values (low initial abstractions occurred for high rain intensities, while high initial abstractions occurred for low rain intensities, in apparent contradiction to theoretical assumptions if initial infiltration was important).

Willeke (1966) examined runoff from four impervious areas in the Newark area. The typical total losses were found to be about 1 mm, but they varied considerably depending on the storm volume. The total storm losses were also found to relate well to surface slope, with decreasing losses and increasing slopes. He attributed this relationship to decreases in micro scale depression storage as the slope increased.

Falk and Niemczynowicz (1978) found that surface storage included an important surface tension term that increased the total depression storage value. Their depression storage value was obtained by taking the depth of rainfall before runoff was observed, considering travel time, from a plot of many different rain and runoff observations. This value therefore included all initial losses, such as flash evaporation and absorption of moisture by dirt. They measured initial abstraction values ranging from 0.13 to 1.75 mm for paved surfaces. The lowest value was for a site having little traffic, while the largest value was for a surface having the "most complicated geometry" with high traffic volumes and deep pools of water along the gutter during rainfall. They also found a correlation between slope and initial abstraction. Their slope initial abstraction relationship was found to vary significantly from the relationships identified by Willeke (1966) and Viessman (1968). These three studies examined the same sites, so the differences in these relationships were thought to be caused by the assumptions used in computing the initial losses. The earlier studies ignored travel time and resulted in generally larger initial losses, while Falk and Niemczynowicz used regression equations between rainfall and runoff and considered travel time. These relationships must be carefully used as they obviously are related to site specific micro-scale detention storage conditions. They concluded that depression storage was considerably smaller than indicated by

other investigators. Lazaro (1979) reported that depression storage may best be determined by calculating actual volumes for small incremental areas and surface roughness heights.

Arnell (1982) measured initial abstractions for mixed urban watersheds in Germany and found initial abstractions varying from 0.38 to 0.7 mm, for watersheds having impervious fractions ranging from 19 to 46 percent. There was a direct relationship between initial abstraction and percent imperviousness, contradicting the usual assumptions that initial abstractions for pervious areas are greater than the initial abstractions for impervious areas.

Pratt and Henderson (1981) examined several European "impervious" test plot data sets and found generally decreasing initial abstraction values (from 1.5 to 0.5 mm) with increasing slope (from 0.5 to 10 percent). However, they also observed much data scatter (the worst case ranged from about 0.1 to 1 mm at about a 2 percent slope) for the paved surfaces, implying the danger of trying to use a general initial abstraction value based only on slope.

Mitchell and Jones (1976) warned that irregular rainfall could decrease the initial abstractions caused by detention storage because of direct inflow from areas having high rainfall intensities filling the storage volumes in areas having low rain intensities. They also noted that for very low rain intensities, detention storage volumes may be emptied by infiltration, resulting in greater initial losses than expected based on surface geometry alone.

Interception Losses

Aron (1982) summarized values for the interception of rainfall by vegetation (from early experiments conducted by Horton) as part of the initial abstraction. Interception can be significant for small rains in areas having large amounts of over-hanging vegetation (such as large mature trees lining older streets). As an example, dense oak trees completely covering a site intercepted about 70 percent of the rain for 2.5 mm rains, about 30 percent of the rain for 13 mm rains, and about 20 percent of the rain for 25 mm rains. Willows intercepted about twice as much rain, while elm and pine trees intercepted about one-half as much rain. Maple and ash trees intercepted rain in about the same manner as oaks, while hickory trees intercepted a fairly constant 1 mm for all rains up to

35 mm. Grasses intercepted much smaller fractions (about 1 to 10 percent) of the rain.

Evaporation Losses

Flash evaporation occurs when rain strikes a hot surface and evaporates on contact or evaporates within the first few minutes after falling as it travels to the drainage system inlet. Longer term evaporation may be responsible (along with infiltration) for depleting detention storage.

Diniz (1980) reported a peak evaporation rate of about 0.8 inches (20 mm) per hour for Austin, Texas. This peak evaporation rate occurred for a brief time in the early afternoon and decreased to almost zero during the night. Grimmond et al. (1986) reported a total peak evaporation potential of about 5 mm per day, and a typical evaporation rate of 1 to 3 mm per day for a Vancouver urban study area. Only about 0.3 mm (or 3 percent) of the rain was lost to evaporation during a typical 3 hour, 10 mm rain.

Evaporation as a direct component of initial abstraction may be small, but Diniz (1980) reported that evaporation may be a significant loss mechanism of ponded water after a storm, especially in arid areas.

Pavement Infiltration Losses

Paved surfaces are usually considered impervious, implying no infiltration. However, some researchers have concluded that paved surfaces do indeed experience infiltration losses. Falk and Niemczynowicz (1978) investigated losses other than surface depression storage losses. They found that smooth paved surfaces had the lowest losses, excluding depression storage (about 0.2 percent of the total rain depth), while poorly maintained surfaces had the largest losses (about 7 percent of the total rain). They therefore concluded that these "other" losses were mostly due to infiltration through the pavement. Pratt and Henderson (1981) were asked after their presentation at the Second International Conference on Urban Storm Drainage if the variation of the runoff coefficient that they observed for pavement could be due to infiltration through the surface which is commonly considered to be zero. They agreed that this variation was likely due to the difference in the permeability of the "impervious" catchment surfaces. They found that gaps between concrete sections in the curbs and gutters were the principal means of runoff losses.

Several other studies have also indicated the importance of cracks in pavement and the resulting infiltration losses. Willeke (1966) found that cracks in gutters could allow significant amounts of water to infiltrate, especially if sandy soils underlaid concrete. Davies and Hollis (1981) found an average runoff loss from a paved road surface to be about 85 percent of the rain depth. This loss was considered about evenly divided between detention storage and infiltration through the pavement, especially through cracks in the gutter.

Cedergren (1974) extensively studied and analyzed infiltration through pavement and through pavement cracks. His studies were directed toward methods to encourage water that has infiltrated through pavement surfaces to pass through pavement bases. Highway and airport engineers are constantly troubled by failures of pavement surfaces because of inadequate drainage of pavement bases. He found that compacted pavement bases of most U.S. roads have very little permeability and therefore little chance of draining completely between rains. He stressed that by 1974, no practical and economical method had been developed that would keep pavements watertight for more than a short period after construction.

Cedergren (1974) conducted infiltration experiments along pavement cracks and found that "very modern" crack sealing procedures were ineffective and that substantial pavement seepage was quite common during and for up to 20 hours after rains. He measured infiltration rates through typical "sealed" joints of about 20 mm per hour (with pavement seams located about every 8 meters).

Cedergren (1974) also examined infiltration through typical pavements. Typical pavement permeability coefficients ranged from a few hundred meters per day (about 10 mm per second) for unsealed asphalt-concrete mixtures down to virtually zero for new, well sealed pavements or older pavements that had been repaved many times. He found that wide expanses of pavements (such as airfields and large parking areas) were more difficult to drain compared to narrower streets because of their relatively large surface area to pavement edge ratios. This was because of the need for pavements to drain through their edges by lateral flows when bases had smaller percolation rates than the pavement itself. Pavement bases were typically less

permeable than the overlying pavement, forcing a general lateral flow pattern. The outflow capability of pavement structures was therefore often less than the inflow rate. The tendency has been for most pavement designers to overestimate the pavement structure outflow rate and to underestimate the inflow rate.

Cedergren (1974) found that typical pavement structures included surface pavements with effective coefficients of permeabilities of about 10 to 100 meters per day being placed over bases with permeabilities of 25 to several thousand times less than these higher rates. The maximum drainage capabilities of many pavement bases could only handle very light drizzles, while many pavements could accept rains having intensities of up to 25 mm per hour. For low permeable bases, the pavement itself acted as detention storage until the base material could drain. With more permeable bases, the drainage was directed downwards and was substantially increased in rate (by several thousand times) compared to typical lateral drainage flow patterns required by slow draining bases.

C. Hypothesized Runoff Loss Model for Impervious Areas

Previous sections summarized literature observations pertaining to runoff losses from impervious surfaces in urban areas and Appendix C contains a discussion of current urban runoff hydrology modeling concepts. A review of this information indicated a need for this dissertation research to verify the use of a simple rainfall-runoff model for impervious areas. The research included detailed homogeneous area tests (Sections 5 and 6) to calibrate the hypothesized general model presented in this subsection. In addition, watershed monitoring (Section 8) verified this model when used in heterogeneous urban drainage areas.

Figure 7.1a shows the hypothesized general model which describes the shape of the relationship between rainfall and runoff. The small-scale tests, reported in Section 5, resulted in runoff information to investigate this proposed model on a small-scale, while the large-scale parking area and flat roof monitoring information, summarized in Section 6, examined paved area runoff responses for much larger areas and for several seasons.

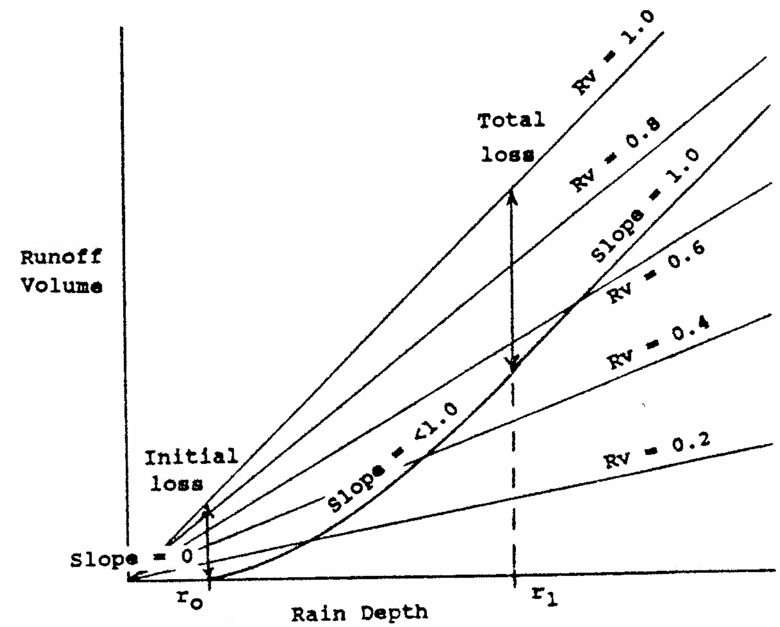


Figure 7.1a Rainfall-Runoff Plot Showing Losses and Rv Values

The runoff response curve shown on Figure 7.1a departs from the x-axis at the rainfall depth when runoff begins (r_0). This depth lag corresponds to initial runoff losses. For impervious areas, initial runoff losses may include flash evaporation, water sorbed by street dirt, surface tension capture due to the scale of roughness, and initial detention storage. The investigations of runoff losses described in Section 5 found that detention storage was by far the most important initial runoff loss mechanism for the paved surfaces investigated. This detention storage volume was usually found to be very closely related to the pavement texture.

After some rain depth (r_1), runoff losses become insignificant. For impervious areas, this is when the detention storage volume becomes filled, evaporation becomes insignificant due to pavement cooling, infiltration through the pavement or through cracks slows practically to nothing, and street dirt becomes saturated.

Between these two rain depths, variable runoff losses occur (assumed to be mostly due to infiltration). The instantaneous variable runoff losses

are relatively large immediately after runoff begins, but decrease as the rain depth increases.

This runoff response curve is shown on Figure 7.1a, superimposed on volumetric runoff coefficient (Rv) lines. The slope of the runoff response line is zero at the beginning of runoff (r_0) and increases to 1.0 (incremental runoff volumes equal incremental rain depths) after all runoff losses are satisfied (at r_1). For small rains, or at the beginning of large rains, the Rv values are very small, but increase dramatically with larger rain depths.

Figure 7.1b shows the model detail describing the variable runoff losses. This figure plots accumulative variable runoff losses (F), ignoring the initial losses, versus accumulative rain (P), after runoff begins. The slope of this line is the instantaneous variable runoff loss (such as infiltration) occurring at a specific rain depth after runoff starts. A polynomial regression equation describing this variable runoff loss model was used in Sections 5 and 6. A theoretical nonlinear model, described next, can also be used to model this relationship.

Two basic model parameters were used to define the model behavior, in addition to rain depth: a, the

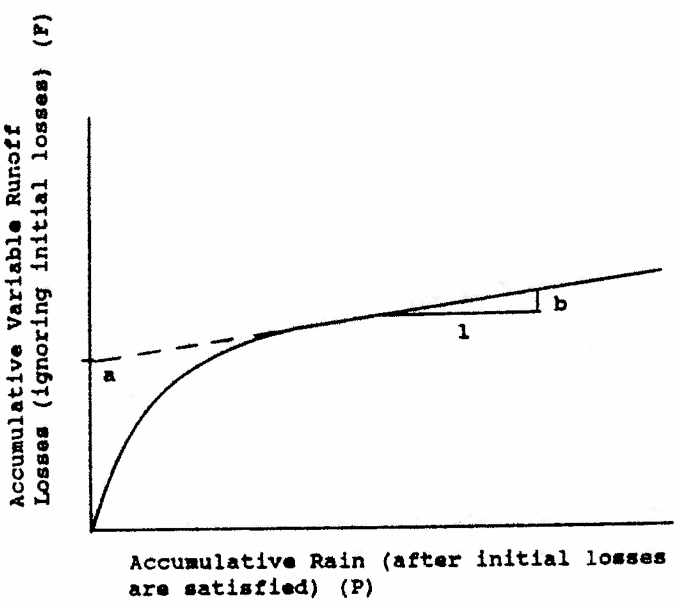


Figure 7.1b Hypothesized Rainfall-Runoff Model Plot

intercept of the equilibrium loss line on the accumulative variable loss axis and b, the rate of the variable losses after equilibrium. If variable losses are zero at equilibrium, then b would be zero. Because this plot ignores initial runoff losses, the variable loss line must pass through the origin of the axes.

For a constant rain intensity (i), total rain depth since the start of runoff (P), equals intensity times the time since the start of runoff (t). The general nonlinear hypothesized model for this variable runoff loss (F) is therefore:

$$F = bit + a(1 - e^{-git}), \text{ or}$$

$$F = bP + a(1 - e^{-gP})$$

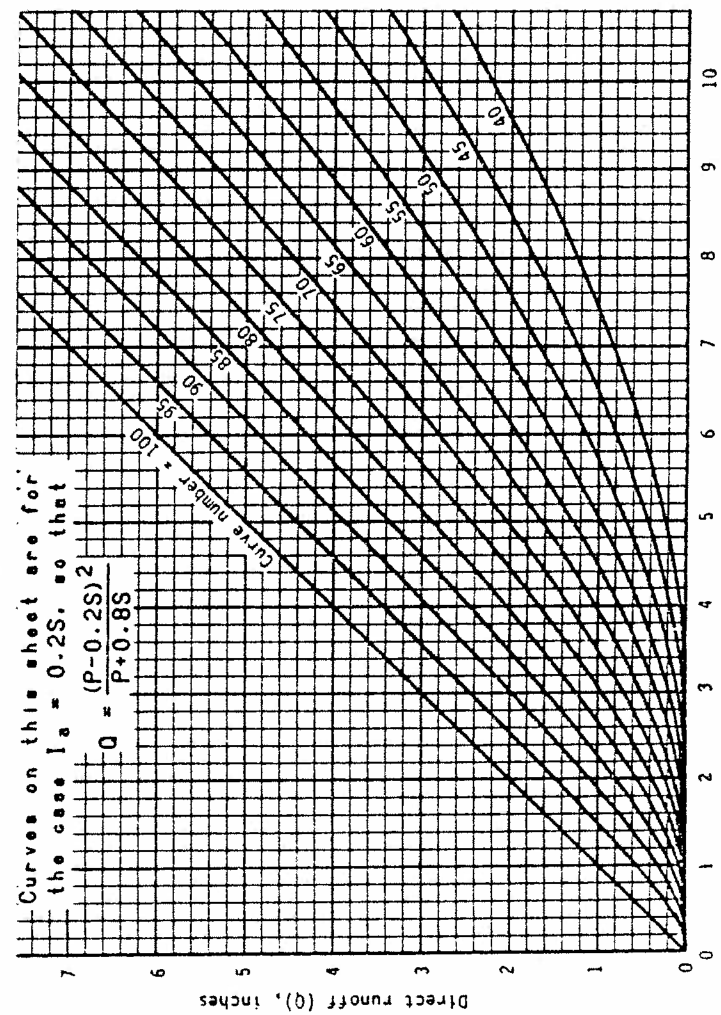
The small- and large-scale paved area runoff data presented earlier were fitted to this equation to determine the values for a, b, and g for observed i and t (or P), and F values as shown later in this Section. The next subsections compare this general model to two common methods used to estimate runoff: the SCS curve number Procedure (SCS 1986) and the Horton infiltration equation (Skaggs et al. 1969).

D. SCS Curve Number Procedure

This dissertation includes discussions of the Soil Conservation Service curve number (SCS CN) procedure (SCS 1986) because of its common use in the design of storm drainage systems and the ability of the hypothesized general model to interface with models using curve numbers. The general model can be used to select curve numbers, allowing the better incorporation of the mutual drainage and flood control benefits of water quality control measures into the design of storm drainage systems.

The SCS CN procedure is commonly used in the design of stormwater drainage systems. Unfortunately, it has some problems when used to evaluate runoff during small storms of interest for water quality analyses. Appendix C describes the development of the SCS CN procedure and reported problems with its use. Figure 7.2 shows the general solution of the SCS equations, relating runoff to rain. This figure is

Figure 7.2 SCS Curve Number Procedure Plot



Source: SCS 1986

similar to Figure 7.1a, but with restrictions. The general SCS CN solution assumes that the ratio of initial abstractions (Ia) to maximum loss (S) is always 0.2. The SCS also assumes that curve numbers remain the same for a watershed, irrespective of the rain depth. Analysis of the test data presented later in this section indicated that these assumptions are not always appropriate.

The most significant problems with the curve number procedure are the common assumptions that curve numbers can be easily selected based on very limited land use information, and that curve numbers remain constant for all rain depths. The curve numbers that are recommended for specific land uses are only applicable for large rains (greater than 25 mm). These curve numbers (and resulting runoff) can be much larger than appropriate for small events which are of most interest in water quality studies. In addition, certain common development practices, such as grass swale drainages and roof disconnections, can greatly reduce runoff discharges, but are not considered in the simple tables commonly used to select curve numbers. Finally, certain water quality control measures, specifically infiltration practices, have dramatic effects on runoff

characteristics and are also not considered in the selection of curve numbers.

The Source Loading and Management Model (Pitt 1986) includes the hypothesized general runoff model, currently calibrated for the Toronto and Milwaukee data, and calculates curve numbers for a variety of rain, site development, and water quality (or quantity) controls. These SLAMM calculated curve numbers can be used in drainage system design models that require curve number values. This results in the consideration of the mutual benefits that some water quality controls have for different drainage system design storms.

The SCS CN procedure was developed as a composite watershed runoff prediction method for large storms, while the hypothesized model was developed for homogeneous paved areas for small storms. When an outfall runoff relationship is developed from individual source area runoff relationships, different source area loss relationships do not perfectly coincide. The concept of partially contributing areas becomes important for most urban areas; the first runoff observed at the outfall originates from the most effectively drained (and/or closest) impervious areas. First runoff does not wait until all initial losses are satisfied for all source areas in the watershed, nor does an average initial abstraction for a complex area accurately predict actual runoff initiation. For large runoff events associated with drainage and flooding studies, these problems may not be significant. For small events of most concern during water quality studies, these additive loss errors can be very important when accurate sources of flows and pollutants need to be known.

The SCS procedure uses a single model parameter (the curve number), besides rain, to predict runoff. As stated in Appendix C, the SCS procedure has become quite popular for predicting runoff for drainage design studies, and has also been used in many water quality studies because of its simplicity and acceptance by the engineering community. The rest of this sub-section examines calculated curve numbers for the paved area Toronto data (along with some paved area Milwaukee data) to demonstrate some of the problems associated with using typical curve numbers for water quality studies. Section 8 summarizes calculated curve numbers for the complex Toronto and Milwaukee monitored watersheds, further demonstrating these SCS CN problems when applied to actual watersheds.

An equation is presented in Appendix C to calculate an observed curve number from precipitation and runoff data, assuming an Ia/S (initial abstraction/maximum runoff loss) ratio of 0.2. The more general equations, derived from the basic SCS equations allowing curve numbers to be calculated using measured initial runoff losses (without the Ia/S assumption), are as follows:

$$S = Ia - P + (P - Ia)^2/Q$$

and $CN = 1{,}000/(S + 10)$

where S, P, Q, and Ia are all given in inches

The initial runoff losses (Ia) directly measured during the Toronto street sheetflow tests and derived from the polynomial equations for the Milwaukee Post Office and Rustler sites (as presented in Sections 5 and 6) are as follows:

Site	Initial Losses (Ia) (mm)	(inches)
Smooth streets	0.44	0.017
Rough streets	0.70	0.028
Milw. Post Office	0.09	0.004
Milw. Rustler	0.88	0.035

These initial runoff losses were used for all runoff observations to calculate runoff curve numbers for the runoff data.

Tables 7.1 and 7.2 show the calculated maximum runoff losses (S) and corresponding curve numbers using the above general curve number equations for the smooth and rough pavement street tests. Figure 7.3 shows plots indicating increasing maximum runoff losses and decreasing CN values as rain depths increase. The mean maximum loss values were 0.10 inches (2.5 mm) for the smooth pavement tests and 0.15 inches (3.7 mm) for the rough pavement tests. The corresponding mean Ia/S ratios were 0.17 for smooth pavement and 0.19 for rough pavement. These ratios were quite similar, indicating little dependence on pavement texture and were very

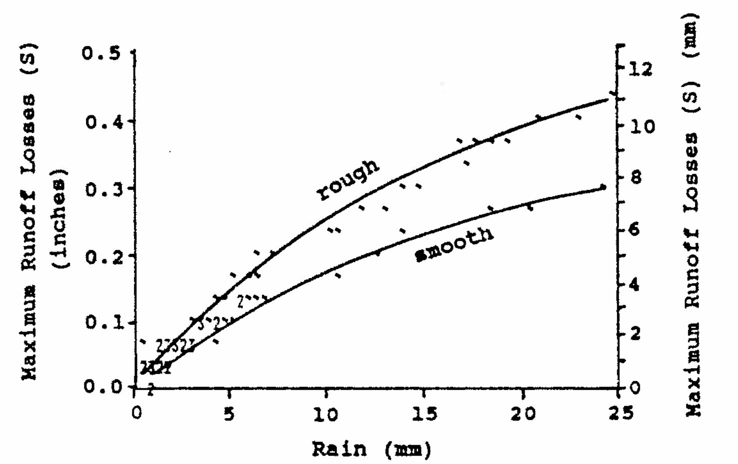
Table 7.1 Observed Maximum Runoff Losses and Curve Numbers for Smooth Street Runoff Tests

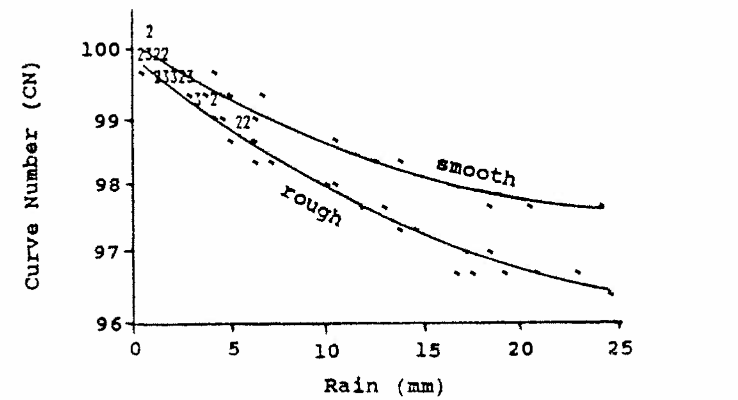
P Rain (in)	Q Runoff (in)	Rv Vol. Runoff Coef.	S Max. Losses (in)	CN Curve Number	"Rain" Intensity (in/hr)	Test Code
0.01	0.00	0.00	--	--	0.47	HCS and HDS
0.03	0.01	0.33	0.02	99	0.47	HCS and HDS
0.09	0.04	0.44	0.05	99	0.47	HCS and HDS
0.16	0.09	0.56	0.08	99	0.47	HCS and HDS
0.23	0.14	0.61	0.11	98	0.47	HCS and HDS
0.40	0.26	0.65	0.17	98	0.47	HCS and HDS
0.48	0.33	0.69	0.20	98	0.47	HCS and HDS
0.54	0.37	0.69	0.21	97	0.47	HCS and HDS
0.71	0.51	0.72	0.25	97	0.47	HCS and HDS
0.80	0.58	0.73	0.26	97	0.47	HCS and HDS
0.94	0.71	0.76	0.27	97	0.47	HCS and HDS
0.01	0.00	0.00	--	--	0.12	LCS
0.01	<0.01	--	<0.01	99	0.12	LCS
0.02	<0.01	--	<0.01	99	0.12	LCS
0.05	0.02	0.40	0.02	99	0.12	LCS
0.07	0.03	0.43	0.02	99	0.12	LCS
0.11	0.06	0.54	0.04	99	0.12	LCS
0.16	0.09	0.56	0.06	99	0.12	LCS
0.19	0.12	0.63	0.07	99	0.12	LCS
0.25	0.16	0.64	0.10	98	0.12	LCS
0.01	0.00	0.00	--	--	0.13	L(D)CS
0.01	<0.01	--	<0.01	99	0.13	L(D)CS
0.03	<0.01	--	<0.01	99	0.13	L(D)CS
0.05	0.01	0.20	0.03	99	0.13	L(D)CS
0.07	0.03	0.43	0.05	99	0.13	L(D)CS
0.13	0.06	0.46	0.09	99	0.13	L(D)CS
0.21	0.12	0.57	0.12	98	0.13	L(D)CS
0.28	0.15	0.54	0.18	98	0.13	L(D)CS

Table 7.2 Observed Maximum Runoff Losses and Curve Numbers for Rough Street Runoff Tests

P Rain (in)	Q Runoff (in)	Rv Vol. Runoff Coef.	S Max. Losses (in)	CN Curve Number	"Rain" Intensity (in/hr)	Test Code
0.03	0.00	0.00	--	--	0.43	HCR
0.05	0.01	0.20	0.03	99	0.43	HCR
0.10	0.04	0.40	0.05	99	0.43	HCR
0.18	0.08	0.44	0.11	98	0.43	HCR
0.25	0.13	0.52	0.15	98	0.43	HCR
0.39	0.23	0.59	0.20	97	0.43	HCR
0.45	0.27	0.60	0.23	97	0.43	HCR
0.54	0.34	0.63	0.26	97	0.43	HCR
0.65	0.40	0.62	0.34	96	0.43	HCR
0.68	0.43	0.63	0.34	96	0.43	HCR
0.76	0.49	0.64	0.34	96	0.43	HCR
0.90	0.61	0.68	0.36	96	0.43	HCR
0.04	0.00	0.00	--	--	0.48	HDR
0.05	0.01	0.20	0.02	99	0.48	HDR
0.08	0.03	0.38	0.04	99	0.48	HDR
0.13	0.06	0.46	0.07	99	0.48	HDR
0.16	0.08	0.50	0.08	99	0.48	HDR
0.21	0.11	0.52	0.11	98	0.48	HDR
0.24	0.14	0.58	0.12	98	0.48	HDR
0.40	0.25	0.63	0.19	98	0.48	HDR
0.51	0.32	0.63	0.24	97	0.48	HDR
0.56	0.35	0.63	0.27	97	0.48	HDR
0.68	0.44	0.65	0.31	96	0.48	HDR
0.72	0.47	0.65	0.33	96	0.48	HDR
0.82	0.54	0.66	0.35	96	0.48	HDR
0.96	0.65	0.68	0.40	96	0.48	HDR
<0.01	0.00	0.00	--	--	0.11	LCR
0.01	<0.01	--	0.14	98	0.11	LCR
0.03	0.01	0.33	<0.01	100	0.11	LCR
0.05	0.02	0.40	<0.01	99	0.11	LCR
0.08	0.03	0.38	0.02	99	0.11	LCR
0.11	0.05	0.45	0.05	99	0.11	LCR
0.14	0.06	0.43	0.07	99	0.11	LCR
0.18	0.10	0.56	0.08	99	0.11	LCR
0.22	0.12	0.55	0.12	98	0.11	LCR
0.02	0.00	0.00	--	--	0.12	LDR
0.03	<0.01	--	<0.01	99	0.12	LDR
0.05	0.01	0.20	0.02	99	0.12	LDR
0.06	0.01	0.17	0.02	99	0.12	LDR
0.07	0.02	0.29	0.04	99	0.12	LDR
0.09	0.03	0.33	0.05	99	0.12	LDR
0.11	0.05	0.45	0.05	99	0.12	LDR
0.13	0.06	0.46	0.06	99	0.12	LDR
0.16	0.07	0.44	0.10	98	0.12	LDR
0.20	0.09	0.45	0.12	98	0.12	LDR
0.24	0.13	0.54	0.12	98	0.12	LDR

Figure 7.3 Maximum Runoff Losses and Curve Numbers Observed for all Street Runoff Tests Combined





close to the SCS assumed ratio of 0.2. However, the SCS assumed Ia/S ratio is used for complex watersheds, not for homogeneous paved areas, and is assumed to be constant for all rains. The maximum runoff losses were greater for rough pavement than for smooth pavement, and showed significant increases with rain depth increases. The overall range for calculated S values was from zero to 0.4 inches. Corresponding Ia/S ratios showed significant decreases (going from >3.0 to 0.07 for the street tests) with increasing rain depths. The curve number values also decreased with increasing rain depths (from about 100 to about 96). Relatively small changes in curve numbers (especially for small rains) can result in significant runoff volume changes. As an example, a 0.25 inch (6.4 mm) rain would produce about 0.25 inches (6.4 mm) of runoff for a curve number of 100, but only about 0.03 inches (0.8 mm) of runoff for a curve number of 95.

Similar calculations were made for S and CN for the large paved and roof areas for the Milwaukee NURP Post Office and Rustler sites. The calculations indicated the same general S and CN trends as the small paved area test data, but these trends were not as clear, probably because of the wider variety of rain

intensity and seasonal conditions included during the Milwaukee monitoring. The mean S values observed for the two sites were relatively close: 0.061 inches (1.6 mm) for the Post Office site and 0.040 inches (1.0 mm) for the Rustler site. However, the Ia values varied greatly for the two sites: 0.004 inches (0.09 mm) for the Post Office site and 0.035 inches (0.88 mm) for the Rustler site. The range of Ia/S ratios for the two sites was also very large, from 0.005 to more than 3.5.

The mean Ia/S ratios therefore also had a large difference: 0.06 for the Post Office site and 0.9 for the Rustler site. The total variable losses appeared to be similar for large paved parking areas and for flat roofs, although the initial abstractions were quite different. The observed curve numbers ranged from about 95 to 100 for both sites, with a slightly smaller range for the Rustler site. As shown previously, this relatively small change in curve number results in very different predicted runoff volumes, especially for small rains.

Tables 7.3 and 7.4 summarize these initial (Ia) and maximum variable losses (S) and the curve numbers calculated using the small and large-scale paved area and flat roof runoff data.

Table 7.3 Summary of Initial Losses and Maximum Losses for Controlled Pavement Sheetflow Tests and Large Milwaukee NURP Test Areas

Test Area	Initial Loss (Ia) mm	Initial Loss (Ia) Inches	Range of Max. Losses (S) from CN Calculations mm	Range of Max. Losses (S) from CN Calculations Inches	Mean Max. Losses (S) from CN Calculations mm	Mean Max. Losses (S) from CN Calculations Inches	Range of Initial Loss (Ia) to Max. Loss (S) Ratio	Ratio of Initial Loss to Mean Max. Loss (Ia/S)
Controlled Pavement Tests:								
Rough pavement	0.78	0.03	<0.25 to 10	<0.01 to 0.40	3.73	0.15	0.08 to >3.0	0.19
Smooth Pavement	0.44	0.02	<0.25 to 6.9	<0.01 to 0.27	2.54	0.10	0.07 to >2.0	0.17
Milwaukee NURP Areas:								
"Post Office"	0.09	0.004	<0.25 to 19	<0.01 to 0.74	1.55	0.06	0.005 to >0.40	0.06
"Rustler"	0.88	0.035	<0.25 to 8.9	<0.01 to 0.35	1.02	0.04	0.10 to >3.5	0.86

Table 7.4 Summary of Observed Curve Numbers for Controlled Pavement Sheetflow Tests and Large Milwaukee NURP Test Areas

Test Area	Number of Observations	Minimum	Maximum	Mean	Standard Deviation
Controlled Pavement Tests:					
Rough pavement	42	96	100	99	1.2
Smooth Pavement	25	97	100	99	0.86
Milwaukee NURP Areas:					
"Post Office"	76	94	100	99	0.94
"Rustler"	65	97	100	100	0.57

E. Horton Infiltration Equation

The Horton infiltration equation, presented in 1939, has been used in many hydrology models (Skaggs et al. 1969). Because of its general acceptance, it will be used in this dissertation as a basic infiltration model for purposes of comparison.

The Horton equation assumes that no runoff will occur until the rain intensity exceeds the infiltration rate of the surface. When the rain intensity exceeds the infiltration rate, runoff will occur (from all areas of the watershed simultaneously). Hydrologists need to know when the infiltration capacity of the soil will be exceeded, and the subsequent decline in infiltration rate as the rain continues (Mein and Larson 1973). The Horton equation is typically presented as:

$$F = F_c + (F_o - F_c)e^{-kt}$$

where F = infiltration rate at any time (t),

F_c = steady state ("final") infiltration capacity,

F_o = initial infiltration capacity (at t=0),

k = a decay constant, dependent on soil and surface conditions.

Horton originally proposed that F_c and k were dependent on the impact energy of falling raindrops (and therefore rain intensity) (Skaggs et al. 1969). In practice, the three- parameter Horton equation is not typically calibrated for different rain intensities. The next subsection discusses this relationship between infiltration and rain intensity.

Urbonas (1979) listed typical Horton equation parameter values for several urban catchments (ranging from 35 to 97 percent imperviousness) in Denver. Equation parameters were determined from fitting observed rainfall-runoff data to the infiltration model. Initial infiltration values were 3 to 4.5 inches per hour, final infiltration values were 0.5 to 1.1 inches per hour, and the k coefficient varied from 0.0007 to 0.0018/sec. It is interesting to note that the airport site (being 97 percent imperviousness) had the same infiltration equation parameters as the site having the lowest percent imperviousness. Terestrip and

Stall (1974) listed various initial and final infiltration rates for different soil conditions. Initial infiltration rates of about 3 inches per hour, final infiltration rates of about 0.1 inches per hour, and k values of about 0.0006/sec were recommended for soils having high runoff potentials (clays with a permanent high water table and a high swelling potential). These Horton equation parameter values for tight soil conditions are close to the parameter values observed in Denver urban areas.

<u>"Problems" with the Horton Equation</u>

Several authors have expressed concern about theoretical problems with the Horton infiltration equation. Some of the most recent questions relate to the Hortonian concept of simultaneous runoff from all areas of the watershed, in contrast to concepts of partial area or variable area contributions (Miller 1984). Aron (1982) also disagreed with the Horton model because it considers infiltration entirely as a function of time rather than soil water storage capacity. Aron also indicated some concern regarding the dependence of the infiltration parameters on rain intensity. He stressed the need for this relationship

between infiltration and rain intensity, but it currently is rarely considered. He also was concerned about the possible effects of variable runoff chemical characteristics that could significantly change the soil permeability during the rain.

Sorooshian and Gupta (1983) questioned the apparent close interaction between the infiltration rate parameters, making calibration very difficult. Viessman et al (1970) found that a Horton type infiltration equation, calibrated using urban area data, failed to accurately predict the early portions of the hydrographs, but did predict the peak flow rates quite well. Brater (1968) stressed the need to obtain rainfall-runoff calibration data from small homogeneous test plots instead of large catchments. Morel-Seytoux (1981) also questioned the way the Horton equation handled ponding before runoff. He stated that the Horton equation only correctly addressed ponding for the limited case when excessive rain intensities start at the beginning of the rain.

F. Comparison of the Hypothesized Model with the Horton Infiltration Equation and the SCS Curve Number Procedure

The Horton infiltration equation has received much attention as a method to predict runoff losses in various urban runoff models. This subsection compares the Horton equation with both the hypothetical general model and the SCS CN procedure, and shows how their parameters are related. The hypothesized model can be directly compared to the Horton infiltration equation. The hypothesized model is:

$$F = bit + a(1 - e^{-git})$$

The total storm infiltration rate is:

$$F = \int F(t)dt$$

where $F(t)$ is an instantaneous infiltration rate.

The instantaneous infiltration rate is then:

$$F(t) = df/dt$$

From the hypothesized model:

$$F(t) = bi + agi(e^{-git})$$

The Horton infiltration equation is:

$$F(t) = Fc + (Fo - Fc)(e^{-kt})$$

where Fc is the final equilibrium infiltration rate,
Fo is the initial infiltration rate,
k is the decay coefficient, and
t is the time since the rain began

Therefore the hypothesized model and the Horton equation are equivalent if the following relationships are simultaneously true:

$$bi = Fc, \text{ or } b = Fc/i$$

$$-git = -kt, \text{ or } g = k/i$$

$$agi = Fo - Fc, \text{ or } a = (Fo - Fc)/gi,$$

$$\text{or } a = (Fo - Fc)/k$$

Rearranging gives:

$$Fc = ib \quad \text{(if Fc is zero, then b is also zero)}$$

$$Fo = ib + aig = i(b + ag)$$

$$k = ig$$

It is seen that the time since runoff began (t) is not a factor in determining any of the Horton infiltration parameters; but rain intensity is a factor, as was previously shown in Section 5.

As was shown on Figure 5.3, the measured accumulative infiltration rates for the high rain intensity tests were much greater than for the low rain intensity tests for the same time since the start of the rain. The infiltration rates (depth per time) were therefore much greater for the high intensity tests. As mentioned in Section 5, the greater infiltration rates with higher rain intensities may have been caused by the greater kinetic energy associated with the high intensity rains (Kinnell 1981). The drops containing high energy must dissipate their energy when striking the ground. The relatively inelastic pavement or

concrete streets (as compared to soil or vegetation) cause small zones of very high stress to occur beneath contacting drops (Springer 1976). Not all of the energy can be readily dissipated by lateral jets from the drops and remaining downward forces may cause increased water penetration into the pavement. These penetration effects may not be as noticeable for soil where deformation of the elastic soil surfaces absorbs much of the drop's energy. Apparent varying infiltration rates for different rain intensities for complex watersheds can also be caused by variable contributing areas, as described later in this subsection.

The SCS CN procedure can also be compared with the hypothesized model and the Horton infiltration equation. The hypothesized model can be rewritten knowing that P = it:

$$F = bP + a(1 - e^{-gP})$$

However, the SCS procedure assumes that the final equilibrium infiltration rate is zero (Fc = 0), therefore b is also zero, leaving:

$$F = a(1 - e^{-gP})$$

When b is zero, the intercept of the runoff loss line is equal to the maximum runoff losses, ignoring initial runoff abstractions (see Figure 7.1b). Therefore, the SCS S' value (maximum variable loss, without Ia) can be substituted for "a" in this equation:

$$F = S'(1 - e^{-gP})$$

There is a distinct relationship between S and CN [CN = 1,000/(S + 10)], and therefore between S' (which is assumed to be equal to 0.8S by the SCS) and CN in the SCS procedure.
Therefore, each curve number has a unique S' value.
Because the SCS CN procedure assumes no final infiltration, the general model b value is zero and the a value is equal to S'. The general model g value was therefore determined using SYSTAT's NONLIN module for the specific F verses P relationships unique for each curve number (and S' value), as shown on Figure 7.4 for curve numbers ranging from 40 to 99. The maximum runoff loss, S', which ignores initial abstractions, occurs after little rain for large curve numbers, but is not reached after more than 125 mm (5 inches) of rain for

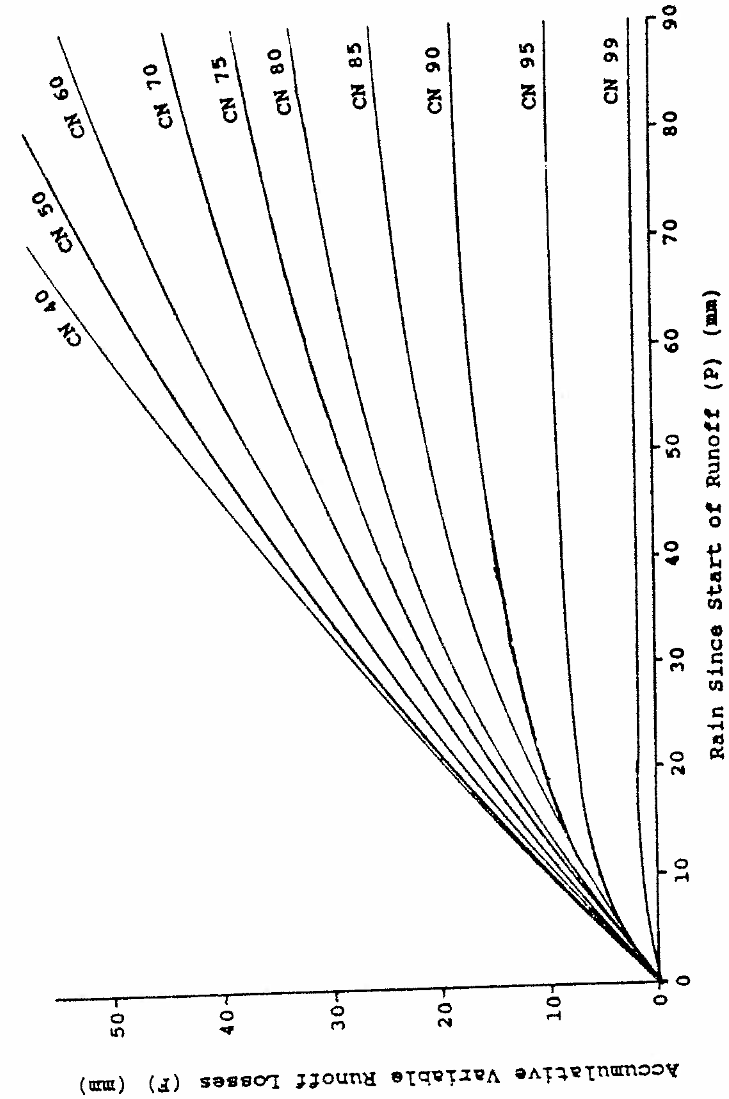


Figure 7.4 Variable Runoff Losses for Different Curve Numbers and Rain Depths

curve numbers less than about 80. Table 7.5 shows the fitted hypothetical equation parameter g values for several curve number values, using SYSTAT's NONLIN module. This table also shows the SCS S' values and the Horton initial infiltration rate and decay coefficients for these curve numbers. According to the controlled street runoff tests and the hypothetical equation, the Horton equation parameters are all related to rain intensity for impervious surfaces, and the hypothetical model g parameter is directly related to the curve number.

Infiltration Relationships with Rain Intensity

In urban hydrology studies, infiltration losses are usually considered to be the most important loss mechanism (Hromadka 1982). The previous discussion shows that infiltration is also an important loss mechanism for pavements, an important topic of this dissertation research. Simple infiltration estimation methods have received much attention in runoff analyses (Singh and Buapeng 1977). Singh and Buapeng found that errors in infiltration estimation may be large and may therefore be responsible for major errors in runoff

Table 7.5 Hypothesized Model and Horton Infiltration Equation Parameters for Different SCS Curve Number Values

Curve Number	Fitted g from hypothesized model	SCS S' Value (ignores Ia) (mm)[1]	Initial Horton Infiltration Rate (Fo) (mm/hr)[2]	Horton Equation Decay Coefficient (k)[3] (1/hr)
99	0.22	2.03	0.45i	0.22i
95	0.042	10.7	0.45i	0.042i
90	0.022	22.6	0.50i	0.022i
85	0.016	35.8	0.57i	0.016i
80	0.012	50.8	0.61i	0.012i
75	0.010	67.8	0.67i	0.010i
70	0.0081	87.4	0.71i	0.0081i
60	0.0057	136	0.78i	0.0057i
50	0.0041	203	0.83i	0.0041i
40	0.0029	305	0.88i	0.0029i

(1) S' = 0.8S assumed by SCS. S' also equals a.
(2) Fo = S'gi, where i equals rain intensity (mm/hr).

Note: The SCS curve number procedure assumes that the final infil tration rate (Fc) is zero.

(3) K = gi, or Fo/S'

predictions. One of the possible sources of infiltration estimation errors is the general lack of consideration of the apparent relationship between infiltration rate and rain intensity. This discussion summarizes some of the literature discussions pertaining to this relationship.

Hawkins (1982) and Kumer and Jain (1982) recognized that infiltration rates vary positively with rainfall intensity; the higher the rain intensity, the higher the infiltration rate. However, few infiltration estimation procedures account for this relationship. Hawkins reported that Chen in 1975 found a relationship between rain intensity and infiltration in the SCS curve number method:

$$\text{loss rate} = i(P/S+0.8)^{-2} \quad \text{(units of in/hr)}$$

where i is the momentary rain intensity. Hawkins also reported how a single storm observation by Hicks in 1938 in the Los Angeles area led to an infiltration and intensity relationship that has been "institutionalized" in Chinese hydrology practice:

$$\text{loss rate} = Ri^{r} \quad \text{(units of mm/hr)}$$

where R (usually > 1) and r (always < 1) are parameters based on site characteristics and antecedent soil moisture conditions.

The relationship between rain intensity and infiltration can be related to the concept of variable contributing areas in heterogeneous watersheds. Areas having low infiltration capacities produce runoff during rains having relatively low intensities, while greater intensity rains are required to produce runoff from areas having high infiltration capacities. Therefore, an overall area infiltration rate appears to be variable and dependent on rain intensity. These variations have not been reported in the literature for homogeneous areas (such as large paved areas). However, as noted previously, infiltration in pavement "systems" includes infiltration through the pavement itself, infiltration through pavement cracks, and infiltration through the pavement base. These different processes would have different infiltration rates; infiltration analysis for the whole system would therefore be intensity dependent.

Hawkins (1982) reported that the rain intensity effects on infiltration have not been observed during rain simulator experiments because almost all rain simulations have been conducted within a relatively narrow range of (high) intensity rains, usually near 75 mm/hr. The test plots would also have to be extremely small (especially for bare ground) to make travel time insignificant. Infiltration ring experimental results have been compared to sprinkler test results to study travel time effects and to examine the effect of "rain" intensities which are always greater than the infiltration rates. The ponding infiltration rings "always" result in greater observed infiltration rates because all areas are being subjected to excessive rain intensities (no variable contributing areas). The effects of the relative head differences of the ponded water versus rain drop impact has not been examined. Wanielista and Yousef (1986) found that infiltration ring experiments resulted in infiltration rates about twice the actual infiltration experienced in flowing roadside grass swales. The static head in the infiltrometers encouraged infiltration more than the "dynamic" head associated with flowing water.

Comparing the Test Data to the Hypothetical General Model and the Horton Infiltration Equation

SYSTAT's NONLIN module was used to estimate the hypothetical general model parameters for the small- and large-scale pavement and large flat roof runoff data, as summarized in Table 7.6. These parameters were estimated with and without final equilibrium infiltration rates to allow comparisons with all SCS CN and Horton equation parameters. The Rustler site data did not allow satisfactory model parameters to be estimated, assuming a final infiltration rate, possibly because this site included a combination of roof and pavement surfaces. Most of the standard errors were substantially less than the estimated parameter calibration values, indicating reasonable model fits. The residuals for all models were examined and the models having the best residual behaviors (Box et al. 1978) are indicated on this table.

Table 7.7 summarizes Horton equation parameter values, using the a, b, and g hypothetical general model parameter values from Table 7.6. The initial infiltration values are quite close for all of the street tests, irrespective of whether the final infiltration rates were assumed to be zero or not.

Table 7.6 **Fitted Parameters for Hypothesized Model for Controlled Small Scale Tests and Milwaukee NURP Sites**

	b	Standard Error For b	a	Standard Error For a	g	Standard Error For g
Rough Street Tests						
With Fc	0.049	0.081	19.6	10.6	0.015	0.005
Assume Fc = 0[(1)]	0 (assumed)	—	14.7	0.84	0.026	0.002
Smooth Street Tests						
With Fc[(1)]	0.12	0.05	3.2	1.6	0.077	0.035
Assume Fc = 0	0 (assumed)	—	9.1	0.94	0.038	0.006
"Post Office" NURP Site						
With Fc[(1)]	0.088	0.037	3.9	0.87	<0.001	0.010
Assume Fc = 0	0 (assumed)	--	36.3	23.8	0.003	0.002
"Rustler" NURP Site						
With Fc	Suspect		Suspect		Suspect	
Assume Fc = 0[(1)]	0 (assumed)	—	1.9	0.41	0.06	0.024

(1) These fitted parameters appeared to have better residual behavior than the alternative set and were therefore preferred.

Table 7.7 **Calculated Horton Equation Coefficients Based on Fitted Parameter Values for Hypothesized Model**

	Initial Infiltration			Final Equilibrium Infiltration			Decay Coefficient		
	Fo = i(b+ag) (mm/hr)	for i=3	for i=11	Fc=bi (mm/hr)	for i=3	for i=11	k=gi (1/hr)	for i=3	for i=11
Rough Street Tests									
With Fc	0.34i	1.02	3.74	0.049i	0.15	0.54	0.015i	0.045	0.17
Assume Fc = 0[(1)]	0.38i	1.14	4.18	0.0	0.00	0.00	0.026i	0.078	0.29
Smooth Street Tests									
With Fc[1]	0.37i	1.11	4.07	0.12i	0.36	1.32	0.077i	0.23	0.85
Assume Fc = 0	0.35i	1.05	3.85	0.0	0.00	0.00	0.038i	0.11	0.42
"Post Office" NURP Site									
With Fc[(1)]	0.09i	0.27	0.99	0.09i	0.27	0.99	<0.001i	<0.003	<0.011
Assume Fc = 0	0.11i	0.33	1.21	0.0	0.00	0.00	0.003i	0.009	0.033
"Rustler" NURP Site									
With Fc	Suspect	–	–	Suspect	–	–	Suspect	–	–
Assume Fc = 0[(1)]	0.11i	0.33	1.21	0.0	0.00	0.00	0.06i	0.18	0.66

1 Preferred coefficient values due to better residual behavior for fitted parameter values.

The effects of the different rain intensities on the Horton equation coefficients are evident from this table. However, even the coefficients associated with the high rain intensity tests (11 mm/hr) are still at least 15 times smaller than the values obtained by Urbonas (1979) from complex urban watersheds in Denver, or the coefficients suggested by Terestrip and Stall (1974) for tight clayey soils.

Variable runoff losses were examined as functions of rain duration (since start of runoff) to determine if the usually assumed Horton equation dependence on rain duration could be substantiated with the street runoff data. Table 7.8 summarizes significant second order polynomial equations (examined using SYSTAT's MGLH module) indicating important, but different, relationships of variable runoff losses (assumed to be infiltration losses) with time for the two sets of street variable runoff loss data arranged by rain intensity. Table 7.9 summarizes the instantaneous infiltration rates observed during these tests, determined from the slopes of these polynomials. The ratios of the infiltration losses for the different time increments (from about 2.2 for the longest durations to about 3.0 for the shortest durations)

Table 7.8 Variable Runoff Losses as Functions of Rain Durations for Controlled Sheetflow Experiments

1) For low intensity rains (average intensity = 3.1 mm/hr):

LOSVAR = 1.44 (duration) - 0.138 (duration)^2
Duration (since start of runoff): 0.00 to 2.12 hr
37 observations
Multiple R^2 = 0.99

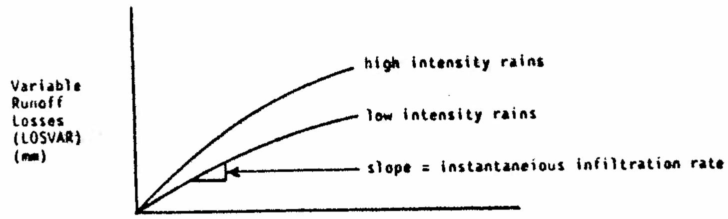
Variable	Coefficient	Standard Error	P(2-Tail)
Duration	1.44	0.078	<0.001
(Duration)^2	-0.138	0.048	0.007

2) For high intensity rains (average intensity = 11.8 mm/hr):

LOSVAR = 4.32 (duration) - 0.581 (duration)^2
Duration (since start of runoff): 0.00 to 2.00 hr
37 observations
multiple R^2 = 1.0

Variable	Coefficient	Standard Error	P(2-Tail)
Duration	4.32	0.16	<0.001
(Duration)^2	-0.581	0.10	<0.001

Table 7.9 Instantaneous Infiltration Rates for Controlled Sheetflow Experiments



1) For low intensity tests:

Slope (LOSVAR) = 1.44 - 0.276 (duration)

2) For high intensity tests:

Slope (LOSVAR) = 4.32 - 1.16 (duration)

Examples:

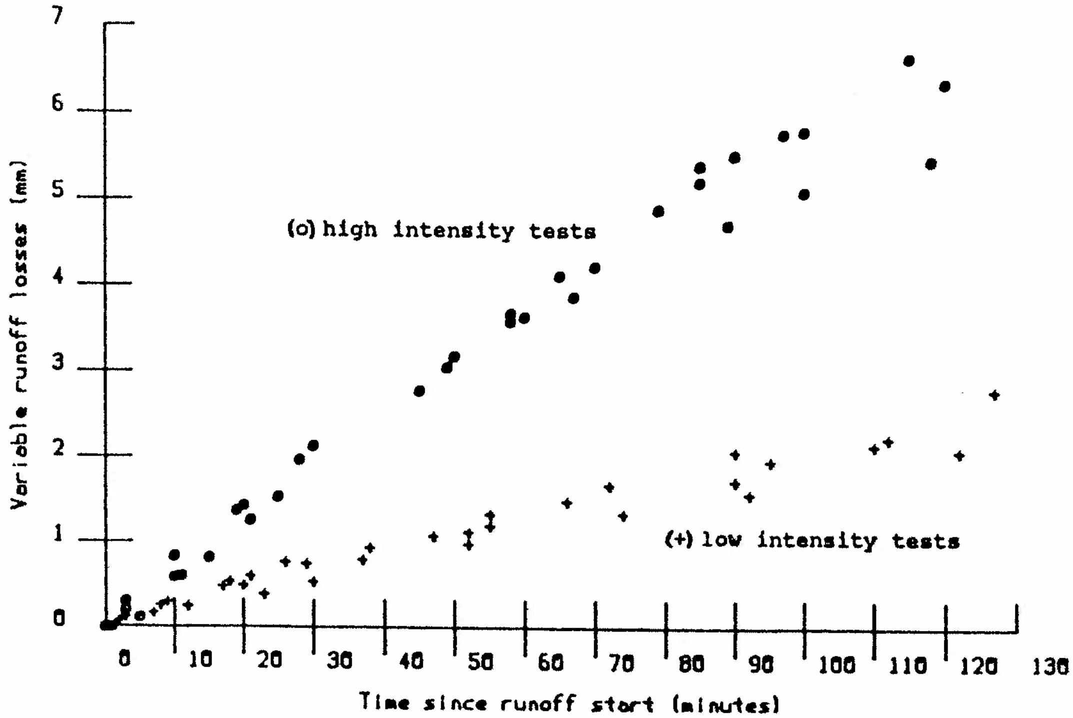
instantaneous infiltration rates (mm/hr) for:

Duration (hrs)	Low Intensity Rains (3.1 mm/hr)	High Intensity Rains (11.8 mm/hr)
0.1	1.4	4.2
0.5	1.3	3.7
0.8	1.2	3.4
1.5	1.0	2.6
2.0	0.9	2.0

appears to be similar to the ratio of the rain intensities used during the runoff tests (about 3.8), possibly indicating greater dependence of infiltration on intensity than on duration. Figure 7.5 (repeated Figure 5.3) shows two plots of the variable runoff losses. Figure 7.5a plots these losses against duration and shows the two distinct trends associated with the different test rain intensities, corresponding to the polynomial equations presented in Table 7.8. Figure 7.5b is a plot of these same variable runoff losses, but against rain depth since runoff began. This figure illustrates the much better (but still not perfect) single relationship that may be obtained using rain depth instead of rain duration as the independent variable in the Horton equation to describe pavement infiltration.

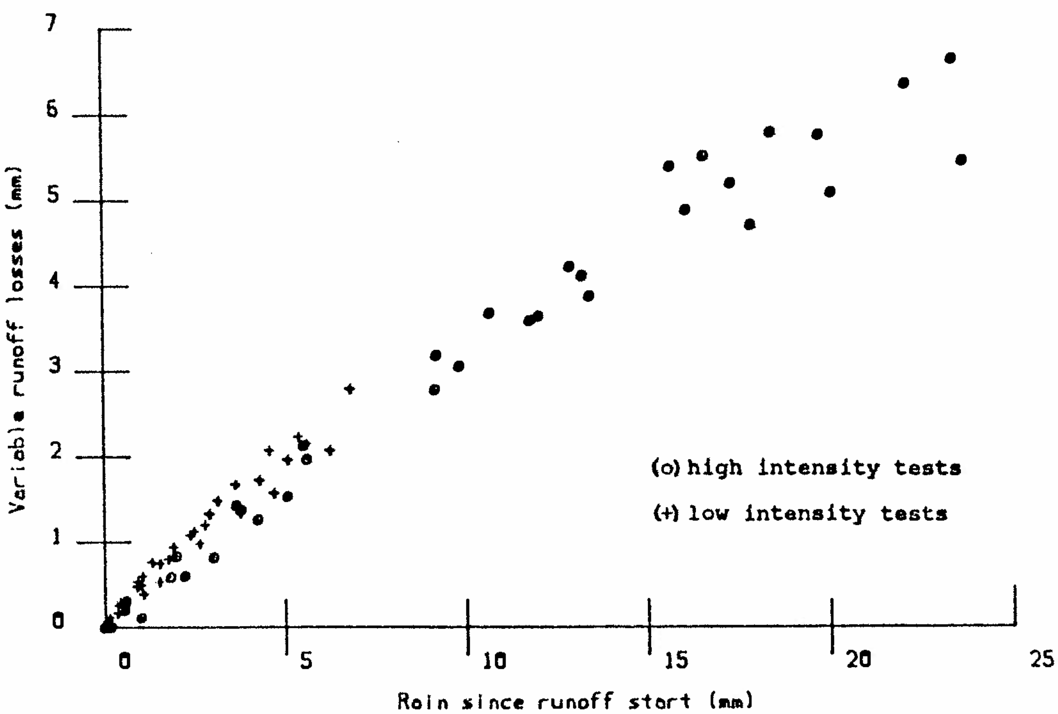
As noted earlier, most runoff models using the Horton infiltration equation use a constant and zero infiltration rate for pavement and only use a detention storage value to describe all pavement runoff losses. Ignoring pavement infiltration (even though it is relatively small) can lead to serious overestimates of pavement runoff yields (by about 25 percent for 25 mm rains). If a runoff model is being calibrated with

Figure 7.5a Variable Runoff Losses by Time



note: duplicate of Figure 5.3a

Figure 7.5b Variable Runoff Losses by Rain Depth



note: duplicate of Figure 5.3b

monitored outfall runoff data, pervious areas of the watershed must have their expected runoff yields incorrectly decreased to compensate for these impervious area runoff estimate errors. Incorrect calculations of source area runoff yields leads to incorrect estimates of effectiveness of many water quality controls and limits the ability of the "calibrated" model to be accurately used in apparently similar watersheds.

Table 7.10 lists the calculated Horton equation parameter values for the street runoff tests. The relationships between the general impervious area runoff model and the Horton equation are seen to be relatively good for the initial infiltration rates. Table 7.7 shows calculated Fo values of 1.02 to 1.14 mm/hr for low intensity (3 mm/hr) rains and Fo values of 3.74 to 4.18 mm/hr for high intensity (11 mm/hr) rains, based on the theoretical relationships between the models. Table 7.10 presents direct NONLIN analysis results of fitting the Horton equation to the data, with similar but generally larger Fo values compared to the previous Fo values (1.6 to 2.1 mm/hr for the low intensity rains and 4.4 to 5.3 mm/hr for the high intensity rains). The Table 7.10 k values are all much

Table 7.10 Fitted Horton Equation Parameters to Observed Infiltration Rates for Controlled Sheetflow Experiments

1) Instantaneous infiltration rate = $Fc + (Fo - Fc)\ exp^{-K\ (Duration)}$

- Could not obtain satisfactory fit for complete Horton equation, therefore assumed Fc = 0 (no long-term, equilibrium infiltration rate).

2) Instantaneous infiltration rate = $Fo\ exp^{-K\ (Duration)}$

	Fo (mm/hr)	Standard Error For Fo	k	Standard Error For k
Low intensity tests:				
LCR	1.6	0.59	0.28	0.46
LDR	2.1	0.39	0.89	0.33
LCS	1.7	0.37	0.61	0.38
L(D)CS	1.8	0.38	0.27	0.32
All low combined	1.8	0.19	0.51	0.18
High intensity tests:				
HCR	4.5	0.65	0.39	0.20
HDR	4.4	0.31	0.21	0.08
HCS	5.3	0.34	0.71	0.10
All high combined	4.7	0.30	0.41	0.09

greater than shown before and do not indicate any trends with rain intensity in contrast to previous predictions.

Table 7.11 summarizes the results of factorial calculations used to identify the significant factors affecting the calibrated parameter values shown on Table 7.10 for initial infiltration and decay coefficient parameters. The final infiltration rates were assumed to be equal to zero for these tests because NONLIN produced final equilibrium infiltration values close to zero that were smaller than the calculated standard error values. The initial infiltration rates were found to be significantly affected by rain intensity alone, while the decay coefficients were not clearly related to any single factor.

G. Calibrated Hypothetical Models for Paved Areas and Large Flat Roofs

Table 7.12 summarizes expected volumetric runoff coefficients and curve numbers for paved areas and large flat roofs for rains from 1 to 125 mm. The street

Table 7.11 Summarized Results for 2^2 Factorial Sheetflow Experiments Examining Horton's Infiltration Rates and Decay Coefficients

1). INITIAL INFILTRATION RATE (F_o) (mm/hr):

$I = 3.01 \pm 0.23$ strong intensity factor

$Y = 3.32 + 1.50\ (I)$

$Y = 1.8$ mm/hr for I- (low intensity rain)

$Y = 4.8$ mm/hr for I+ (high intensity rain)

2) DECAY COEFFICIENT (k) (hr^{-1}):

$IT = -0.29 \pm 0.18$ weak intensity and texture interaction
$IC = -0.30 \pm 0.20$ weak intensity and cleanliness interaction

$Y = 0.51 - 0.15\ (IT) - 0.15\ (IC)$

$Y = 0.21$ for IT+ and IC+

$Y = 0.51$ for IT+ and IC-

$Y = 0.51$ for IT- and IC+

$Y = 0.81$ for IT- and IC-

Table 7.12 Rainfall-Runoff Relationships for Paved Areas and Roofs

Rain (mm)	Rough Streets runoff (mm)	Rough Streets Rv	Rough Streets Calculated CN	Smooth Streets runoff (mm)	Smooth Streets Rv	Smooth Streets Calculated CN	All Streets Combined runoff (mm)	All Streets Combined Rv	All Streets Combined Calculated CN	Large Paved Parking[1] runoff (mm)	Large Paved Parking[1] Rv	Large Paved Parking[1] Calculated CN	Large Connected Flat Roofs[2] runoff (mm)	Large Connected Flat Roofs[2] Rv	Large Connected Flat Roofs[2] Calculated CN
1	0.2	0.18	99	0.4	0.35	100	0.3	0.26	100	0.9	0.93	100	0	0	-
2	0.8	0.39	99	1.0	0.49	99	0.9	0.43	99	1.9	0.96	100	0.005	0.002	-
3	1.4	0.47	99	1.6	0.54	99	1.5	0.49	99	2.9	0.96	100	0.9	0.30	99
5	2.7	0.53	99	3.0	0.59	99	2.7	0.55	99	4.9	0.97	100	2.7	0.54	99
10	6.0	0.60	98	6.5	0.65	98	6.1	0.60	99	9.7	0.97	100	7.2	0.72	99
15	9.5	0.64	98	10.3	0.69	98	9.6	0.64	98	14.6	0.97	100	11.8	0.79	99
20	13.3	0.67	97	14.4	0.72	98	13.4	0.67	97	19.3[3]	0.97	100	16.6	0.83	99
25	17.4	0.70	97	19.0[3]	0.76	98	16.8[3]	0.67	97	24.3	0.97	100	21.1	0.84	99
30	21.8[3]	0.73	97	24.0	0.80	98	21.8	0.73	97	29.3	0.98	100	25.7	0.86	98
40	31.8	0.80	97	34.0	0.85	98	31.8	0.80	97	39.3	0.98	100	35.1	0.88	98
50	41.8	0.84	97	44.0	0.88	98	41.8	0.84	97	49.3	0.99	100	44.8[3]	0.90	98
60	51.0	0.86	97	54.0	0.90	98	51.8	0.86	97	59.3	0.99	100	54.8	0.91	98
70	61.8	0.88	97	64.0	0.91	98	61.8	0.88	97	69.3	0.99	100	64.8	0.93	98
80	71.8	0.90	97	74.0	0.93	98	71.8	0.90	97	79.3	0.99	100	74.8	0.94	98
90	81.8	0.91	97	84.0	0.93	98	81.8	0.91	97	89.3	0.99	100	84.8	0.94	98
100	91.8	0.92	97	94.0	0.94	98	91.8	0.92	97	99.3	0.99	100	94.8	0.95	98
125	117	0.93	97	119	0.95	98	117	0.93	97	124	0.99	100	120	0.96	98

[1] Unpaved parking areas are assumed to have 6 mm initial losses and the same infiltration losses as paved parking areas.

[2] Pitched connected roofs are assumed to have 0.75 mm initial losses and no other losses.

[3] Constant runoff losses occur after these rain volumes.

runoff curves were extrapolated from the controlled street runoff tests for this larger range of rains. These data were then used with the Milwaukee NURP Post Office site data to estimate large paved parking area runoff responses by difference. These street and parking area estimates were then used with the Rustler data to estimate the large flat roof runoff responses. The maximum total losses and the rain at which these losses occurred were estimated from polynomial regression equations (as shown on Table 5.9 for the streets and Tables 6.8 and 6.9 for the two Milwaukee impervious test areas). The street runoff experiments only involved rains up to 25 mm, but the Milwaukee NURP data included a few very large events. The street runoff response curves were extrapolated beyond the available small-scale test data range, using the large event Milwaukee data. This resulted in different equation parameters than if the narrower range of data was used, as are shown on Table 7.8. Figures 7.6 through 7.10 plot the resultant rainfall-runoff curves and curve number changes with rain for the different impervious surfaces evaluated.

Table 7.13 summarizes the hypothetical model, SCS curve number procedure, and Horton equation coefficient

Figure 7.6 Rainfall-Runoff and Curve Numbers for Rough Textured Streets

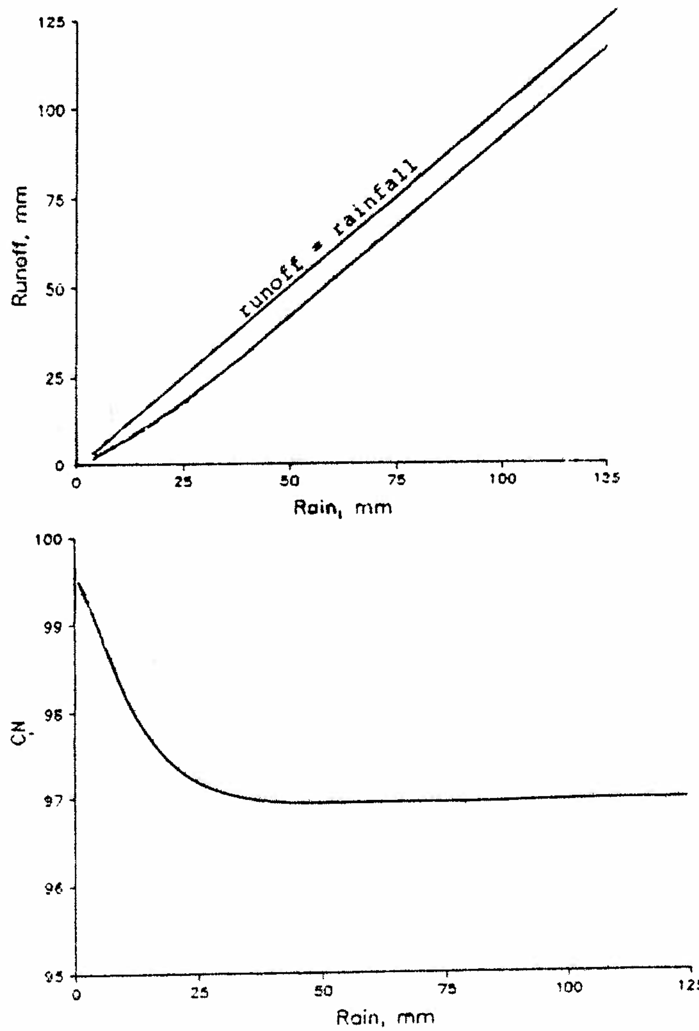


Figure 7.7 Rainfall-Runoff and Curve Numbers for Smooth Textured Streets

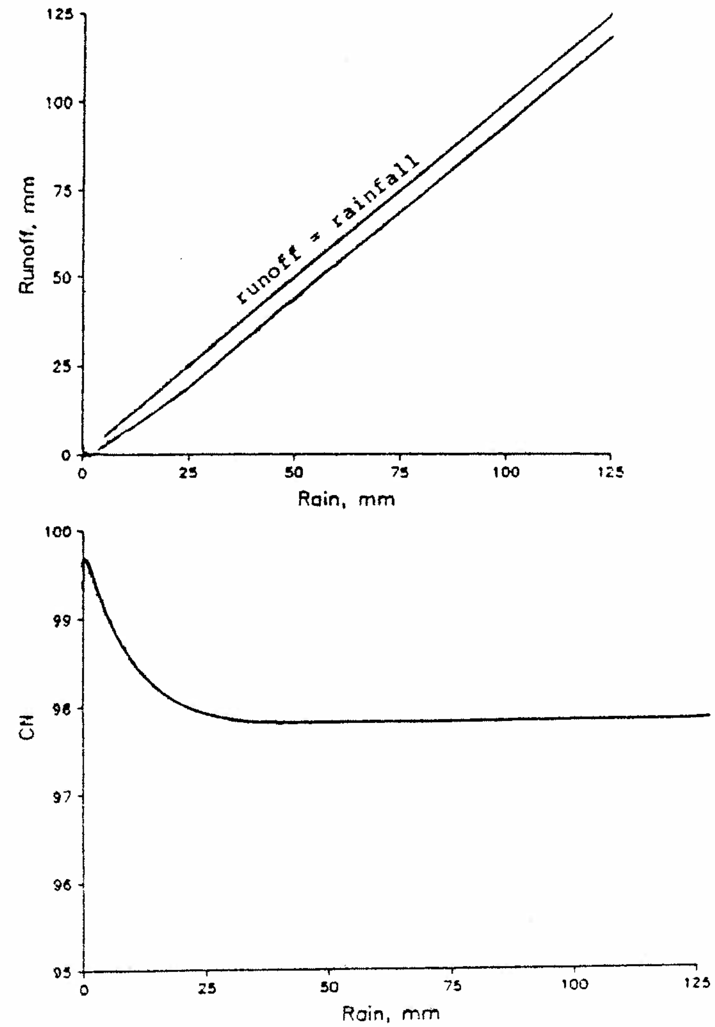


Figure 7.8 Rainfall-Runoff and Curve Numbers for Intermediate Textured Streets

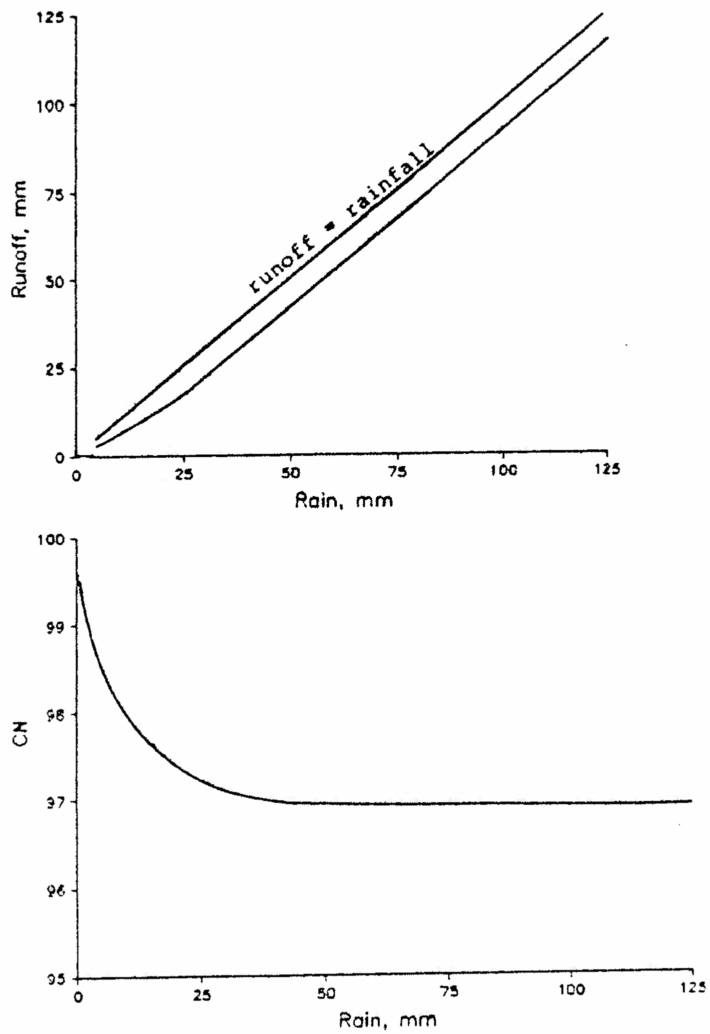


Figure 7.9 Rainfall-Runoff and Curve Numbers for Large Paved Parking Areas

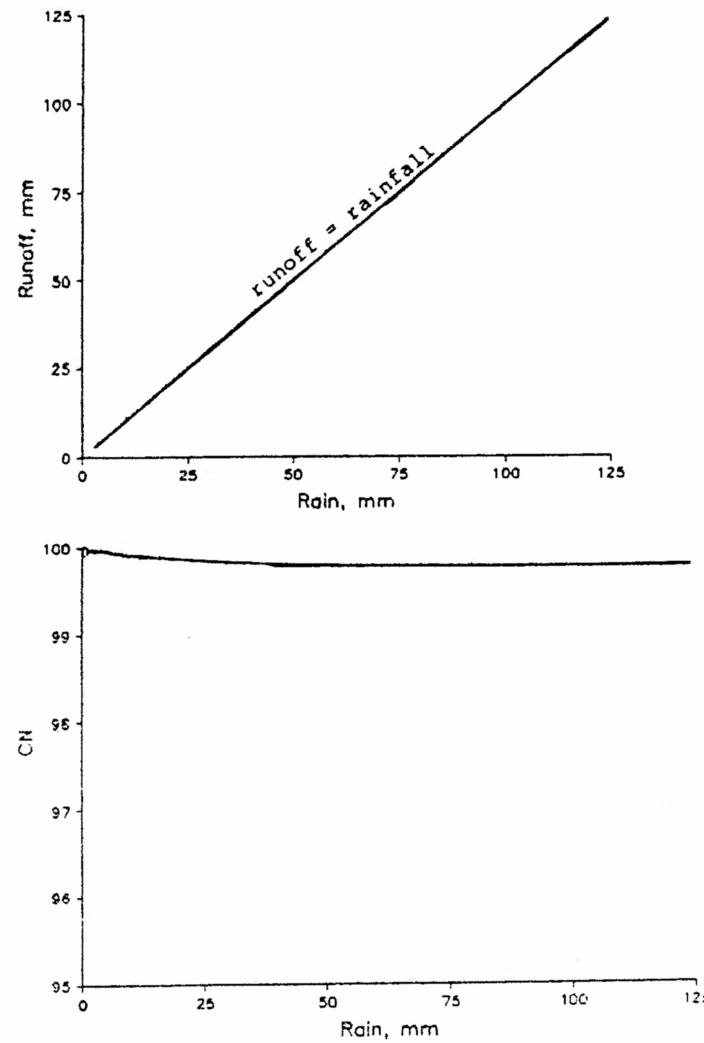


Figure 7.10 Rainfall-Runoff and Curve Numbers for Flat Connected Roofs

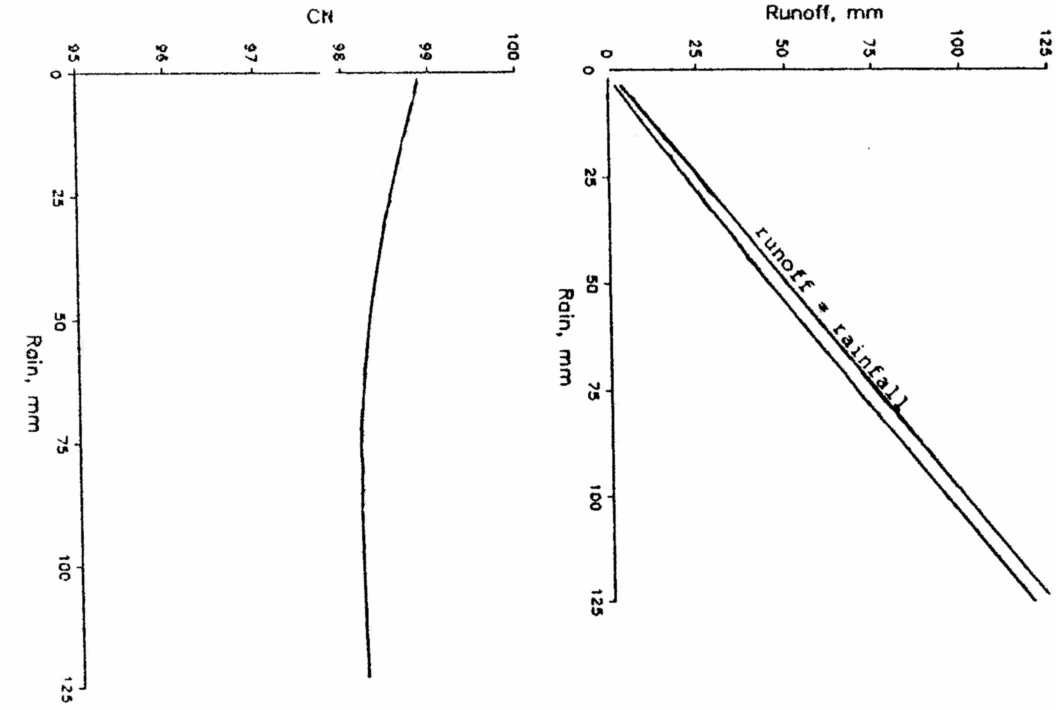


Table 7.13 Equation Coefficient Values for Impervious Surface Hydrology Models (for 1 to 125 mm rains)

Homogeneous Impervious Surface:	Calibrated General Model Coefficients[1] a	Calibrated General Model Coefficients[1] g	SCS Curve Number Procedure: Initial Losses (Ia) (mm)	SCS Curve Number Procedure: Estimated Maximum Total Losses (S) (mm)	SCS Curve Number Procedure: Estimated Rain When Max. Total Losses Occur (mm)	Horton Equation: Initial Infiltration Rate (Fo) (mm/hr)	Horton Equation: Proportionality Constant (k) (1/hr)
Rough streets	7.7	0.072	0.70[2]	8.2	41	0.55i	0.072i
Smooth streets	5.7	0.093	0.44[2]	6.0	32	0.53i	0.093i
All streets combined	7.9	0.074	0.55[2]	8.2	41	0.58i	0.074i
Large paved parking area	0.67	0.075	0.03	0.7	18	0.050i	0.075i
Large flat roof	1.93	0.040	1.93	5.2	51	0.14i	0.040i

(1) Final equilibrium infiltration rates were found to be very small, resulting in insignificant (and very small) B parameter coefficients.

(2) These initial losses were directly measured during the sheetflow experiments, while the other Ia values along with the S and "rain when S occurs" values were estimated from polynomial regressions of the data.

values for these impervious surfaces. These model coefficients represent estimated conditions for the range of rains from 1 to 125 mm. The final equilibrium infiltration values were found to be quite small, and significant b coefficients could not be determined. These curves and these model parameter coefficients therefore assume no final infiltration, resulting in b values of zero. The maximum total runoff losses (S) should therefore be equal to the a coefficient values, as shown on Figure 7.1b. The S and b values shown on this table are reasonably similar (as predicted), with the largest differences found for the large flat roofs.

The runoff losses from the large paved parking areas were less than for the street runoff losses, probably because of the geometry of the pavement structure. Cedergren (1974) found that large paved areas were less well drained than smaller areas because the large areas had smaller ratios of pavement edges to surface areas. He found that pavements mostly drain laterally through the pavement itself, not vertically through the pavement base. These data possibly support this earlier observation.

The large flat roofs also experienced much larger initial losses than the paved areas, possibly because

of the very rough textures of the roofs (large rocks placed on uneven tar) allowing several mm of rain to be lost as initial detention storage, and ultimately through evaporation. Roofs also experience very little infiltration losses (in the assumed absence of roof leaks), also leaving evaporation losses as the major variable runoff loss mechanism.

H. Comparison of Source Area Contributions Using SCS CN, Horton, and General Impervious Area Models

Runoff models are typically calibrated using outfall hydrology data and information concerning the watershed's characteristics. The SCS curve number procedure, the Horton equation, and the general model developed during this research require at least the areas of different types of watershed surfaces in order to be calibrated with outfall hydrology data. Additional information may also be needed, especially concerning the type of stormwater drainage system, roof drainage connections, the types of impervious areas, and soil infiltration characteristics. Section 10 discusses the use of the general model and its incorporation into the Source Loading and Management Model (SLAMM). This subsection is a brief comparison of the results obtained from these models to estimate the relative contributions of impervious and pervious areas, using the same outfall data.

The effects of different treatments of impervious area runoff were investigated by using the outfall hydrology and watershed characteristic data of the mixed residential and commercial Thistledowns watershed, as presented in Section 8. The different impervious area modeling procedures were used to estimate the directly connected impervious area runoff contributions. These contributions were then subtracted from the total outfall runoff observations to obtain the runoff contributions from the other areas. The relative contributions from the directly connected impervious areas and the other areas were then compared for the different modeling procedures.

A constant curve number of 98 was used for the directly connected impervious areas when using the SCS procedure. No impervious area infiltration was assumed when using the Horton equation approach, but two different initial abstractions (1.0 and 1.6 mm) were used. The general model results were obtained from the

analysis to be presented in Section 8. Analysis of runoff contributions from directly connected impervious and all other areas was conducted for 5, 10, 25, and 50 mm rains, as shown on Table 7.14.

The Thistledowns volumetric runoff coefficient (Rv) varied from about 0.17 to 0.21 for this watershed and for this range of rains. The directly connected impervious area contributions are shown in Section 8 to range from a high of 96 percent for the smallest rain to about 85 percent for the largest rain when using SLAMM. This reflects the decreasing importance of paved areas as the rainfall depth increased. However, the SCS and Horton modeling approaches resulted in increased contributions of runoff from impervious areas as rainfall depth increased. For the largest rain evaluated, the SCS and Horton modeling approaches also resulted in substantially greater runoff contributions from paved areas, compared to SLAMM. Except for the Horton approach using 1.0 mm of initial abstractions, the other approaches also resulted in less runoff from impervious areas during the smallest rains as compared to SLAMM. If only 1.0 mm initial abstractions for directly connected impervious areas were assumed when using the Horton approach, the paved areas would be

Table 7.14 Flow Contribution Estimates from Source Areas for Different Prediction Procedures (Thistledowns mixed residential and commercial area)

Rain (mm)	Observed Runoff (mm)	SCS CN Procedure imperv. (%)	SCS CN Procedure pervious (%)	Horton Infiltration Equation, 1.6 mm Detention Storage for Imperv. imperv. (%)	Horton Infiltration Equation, 1.6 mm Detention Storage for Imperv. pervious (%)	Horton Infiltration Equation, 1.0 mm Detention Storage for Imperv. imperv. (%)	Horton Infiltration Equation, 1.0 mm Detention Storage for Imperv. pervious (%)	General Impervious Area Runoff Model imperv. (%)	General Impervious Area Runoff Model pervious (%)
5	0.9	84%	16%	86%	14%	100%	0%	96%	4%
10	2.0	84	16	90	10	100	0	95	5
25	5.2	83	17	98	2	100	0	92	8
50	10.6	92	8	100	0	100	0	85	15

assumed to contribute all of the runoff during all events. Obviously, selection of an initial abstraction value (even over a small range) can have a significant effect on the predicted relative runoff contributions from different source areas.

These different runoff contribution predictions caused by different modeling procedures can have significant effects on the predictions of source area runoff volume and pollutant controls, even though they all may result in comparable outfall predictions. When the calibrated models are used in other watersheds having different source areas, the outfall predictions from the different models could also vary significantly. Accurate source area runoff predictions are therefore necessary in order to predict both the outfall and source area volume and pollutant contributions and the effects of different control measures.

I. Summary of Hydrology Model Relationships

This section presented and examined a hypothesized general runoff model for impervious areas in relationship to the SCS curve number procedure and the Horton infiltration equation. The general model was based on data from small- and large-scale impervious area tests presented in Sections 4 through 6. The model was then calibrated using the street runoff experimental data and selected large-scale impervious area data obtained during the Milwaukee NURP project.

An important finding of this research was that the model can be used to develop reasonable SCS curve numbers that reflect different developmental conditions, water quality controls, and different rain depths. These modified curve numbers can then be used in the design of drainage systems reflecting the significant water volume benefits of many water quality controls. The relatively small increases in curve number values observed for decreasing rain depths actually reflect very significant runoff volume increases for rains less than about 25 mm (the rains of most concern for most water quality analyses). Therefore, the casual selection and use of curve numbers can create substantial errors when predicting sources of pollutants and seasonal pollutant yields. Again, the use of modeling procedures developed for

flood and drainage studies can be inappropriate for water pollution studies.

This research demonstrated that the Horton equation parameters can be directly related to the general paved area model parameters and to SCS curve numbers by rain intensity. The parameters of the Horton infiltration equation for paved surfaces were found to vary significantly depending on rain intensity (infiltration increased with increasing rain intensity) and not rain duration during the street sheetflow tests. The Horton parameters were not constant for different rain intensities as is generally assumed by users of the Horton equation.

The three runoff prediction procedures examined in this section all resulted in different source area contribution estimates for the same set of outfall calibration data. These differences could result in significant errors when predicting the effects of outfall and source area pollutant and volume controls, especially when using calibrated models in different watersheds.

The next section uses this calibrated impervious area model in complex urban watersheds. The response of pervious areas is estimated using low density urban area outfall data. The calibrated overall model is then verified by examining individual events from the Toronto test watersheds and from random monitored Milwaukee events.

SECTION 8

VERIFICATION OF HYPOTHESIZED PAVED AREA HYDROLOGY MODELS AND THEIR USE IN COMPLEX URBAN WATERSHEDS

A. Introduction

An objective of this section is to verify the previously described and calibrated impervious area hydrology models using runoff observations from complex urban watersheds. Another objective of this section is to show how these models can be used to identify significant flow sources needed for selecting runoff volume controls and to calculate appropriate curve numbers for drainage system designs.

Selected outfall runoff observations were used to estimate pervious area effects, after subtracting the impervious area runoff responses. Random individual events from eight Milwaukee test watersheds and all of the Toronto test watershed runoff observations were then used in the composite urban runoff model for verification. Finally, the verified model was used in example calculations to show flow contributions from different source areas and how flow reduction benefits of water quality controls could be used to produce adjusted curve numbers for designing drainage systems.

B. Test Watershed Descriptions and Outfall Runoff Responses

Runoff data from the two Toronto test watersheds (Emery, an industrial area, and Thistledowns, a mixed residential/commercial area described in Appendix B), along with data from eight Milwaukee NURP test watersheds (Bannerman et al. 1983), were used to verify the general impervious area hydrology model. Table 8.1 summarizes the land surface characteristics of these ten watersheds. The watershed characteristics varied significantly, with directly connected impervious areas ranging from about 20 to 100 percent of the complete watersheds. Partially disconnected impervious areas made up from less than 0.1 to more than 30 percent of the watersheds' areas. The remainder of the test watersheds were pervious surfaces (<0.1 to 55 percent).

Table 8.2 and Figures 8.1 and 8.2 illustrate the smoothed outfall runoff responses (from regressions of

Table 8.1 Land Surface Characteristics of Test Watersheds (Percent)

	Toronto Test Areas		Milwaukee NURP Monitored Areas							
	Indus.	Resid./Commer.	Medium Den. Resid.		Medium Den. Resid. (with Alleys)		Mixed Strip Commer. and High Density Resid. (With Alleys)		Shopping Center	
	Emery	Thistledowns	North Hastings	Burbank	West Congress	Lincoln	State Fair	Wood Center	Post Office	Rustler
Directly Connected Impervious Areas										
Rough Streets	1.2	0.6	<0.1	<0.1	<0.1	<0.1	8.2	<0.1	<0.1	<0.1
Smooth Streets	3.9	5.0	<0.1	7.5	<0.1	<0.1	8.2	3.9	<0.1	<0.1
Intermediate Streets	0.6	3.9	13.1	7.5	16.1	20.5	8.1	23.3	11.0	10.0
Connected Flat Roofs	31.1	<0.1	<0.1	<0.1	<0.1	<0.1	10.2	13.3	<0.1	42.0
Connected Pitched Roofs	<0.1	7.8	5.2	4.5	3.9	3.9	10.1	3.4	<0.1	<0.1
Other Con. Impervious[1]	5.3	3.0	11.6	11.8	13.1	15.5	24.8	28.7	86.0	48.0
	42.1	21.2	29.9	30.5	33.1	39.9	69.6	72.6	97.0	100.0
Partially Disconnected Impervious Areas (draining to pervious areas)	32.6	23.3	21.5	19.5	17.2	17.4	7.6	8.5	<0.1	<0.1
Pervious Areas	25.3	55.1	47.9	49.4	49.7	42.6	22.8	18.9	3.0	<0.1
Disconnected Areas	<0.1	0.4	0.7	0.6	<0.1	0.1	<0.1	<0.1	<0.1	<0.1
Totals	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

[1] Includes paved parking and storage areas, plus other paved areas that have storm drain inlets or that drain by sealed waterways to the storm drainage system.

Table 8.2 Smoothed Outfall Rainfall-Runoff Observations for Toronto Test Watersheds and Milwaukee NURP Monitored Watersheds

	Toronto-Emery (Industrial)			Toronto-Thistledowns (Resid./Commercial)			Milwaukee-medium density resid.		
Rain (mm)	runoff (mm)	Rv	calculated CN	runoff (mm)	Rv	calculated CN	runoff (mm)	Rv	calculated CN
1	0.09	0.09	99	<0.01	<0.01	-	<0.01	<0.01	-
2	0.37	0.19	99	0.22	0.11	98	0.25	0.12	98
3	0.65	0.22	98	0.44	0.15	98	0.60	0.20	98
5	1.2	0.24	97	0.87	0.17	97	1.3	0.26	97
10	2.6	0.26	95	2.0	0.20	94	3.0	0.30	96
15	4.0	0.27	93	3.0	0.20	91	4.9	0.33	94
20	5.4	0.27	91	4.1	0.21	89	6.8	0.34	92
25	6.8	0.27	89	5.2	0.21	87	8.8	0.35	91
30	8.2	0.27	87	6.3	0.21	84	10.9	0.36	90
40	11.0	0.27	83	8.5	0.21	80	15.3	0.38	87
50	13.8	0.28	80	10.6	0.21	77	20.1	0.40	85
60	16.6	0.28	77	12.8	0.21	73	25.2	0.42	84
70	19.3	0.28	74	15.0	0.21	70	30.7	0.44	82
80	22.1	0.28	71	17.2	0.21	67	36.4	0.45	81
90	24.9	0.28	69	19.3	0.21	65	42.4	0.47	80
100	27.7	0.28	67	21.5	0.21	62	48.9	0.49	79
125	34.7	0.28	62	26.9	0.22	57	66.3	0.53	78

	Milwaukee-medium density resid. with alleys			Milwaukee-commercial and high density resid.			Milwaukee-shopping center		
Rain (mm)	runoff (mm)	Rv	calculated CN	runoff (mm)	Rv	calculated CN	runoff (mm)	Rv	calculated CN
1	0.14	0.14	99	<0.01	<0.01	-	<0.01	<0.01	-
2	0.50	0.25	99	0.64	0.32	99	0.43	0.22	99
3	0.89	0.30	99	1.4	0.47	99	1.5	0.48	99
5	1.7	0.33	98	3.0	0.59	99	3.4	0.68	99
10	3.7	0.37	96	6.7	0.67	99	8.1	0.81	99
15	6.1	0.41	96	10.8	0.72	98	13.1	0.87	99
20	8.7	0.44	94	15.1	0.75	98	18.0	0.90	99
25	11.5	0.46	93	19.2	0.77	98	22.7	0.91	99
30	14.6	0.49	93	23.8	0.79	98	27.7	0.92	99
40	21.4	0.54	92	33.1	0.83	98	37.4	0.94	99
50	29.4	0.59	91	43.1	0.86	98	47.3	0.95	99
60	38.2	0.64	91	53.1	0.89	98	56.9	0.95	99
70	48.2	0.69	92	63.1	0.90	98	66.0	0.95	99
80	58.2	0.73	92	73.1	0.91	98	76.5	0.96	99
90	68.2	0.76	92	83.1	0.92	98	86.1	0.96	99
100	78.2	0.78	92	93.1	0.93	98	96.1	0.96	99
125	103	0.83	92	118	0.94	98	120	0.96	98

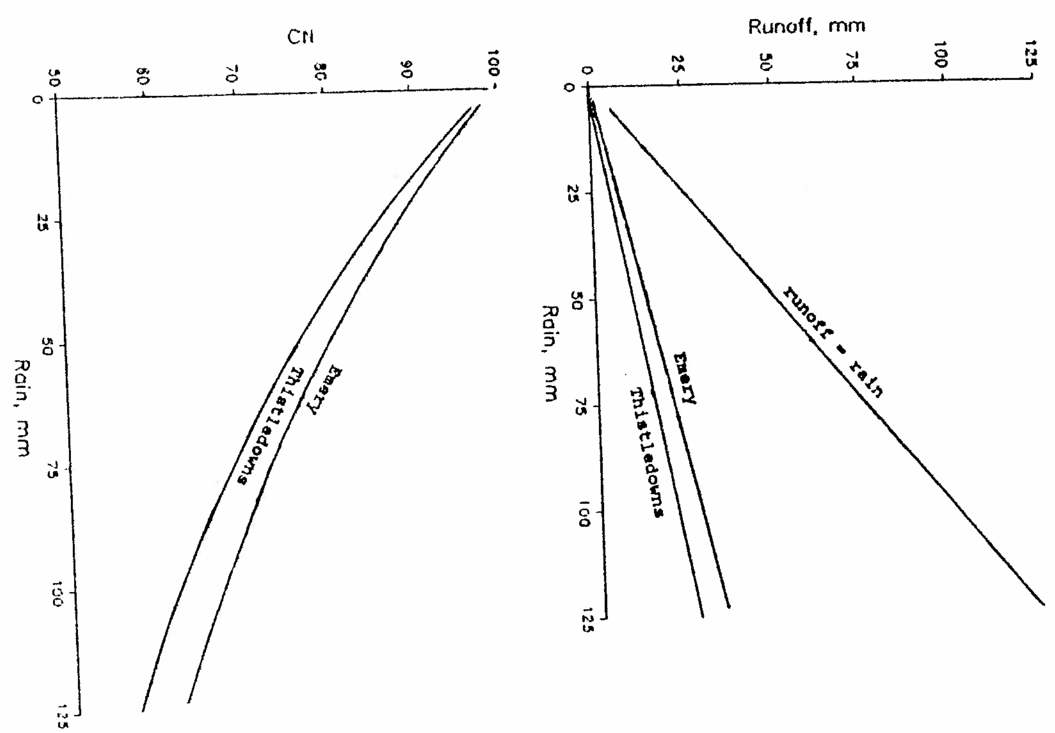
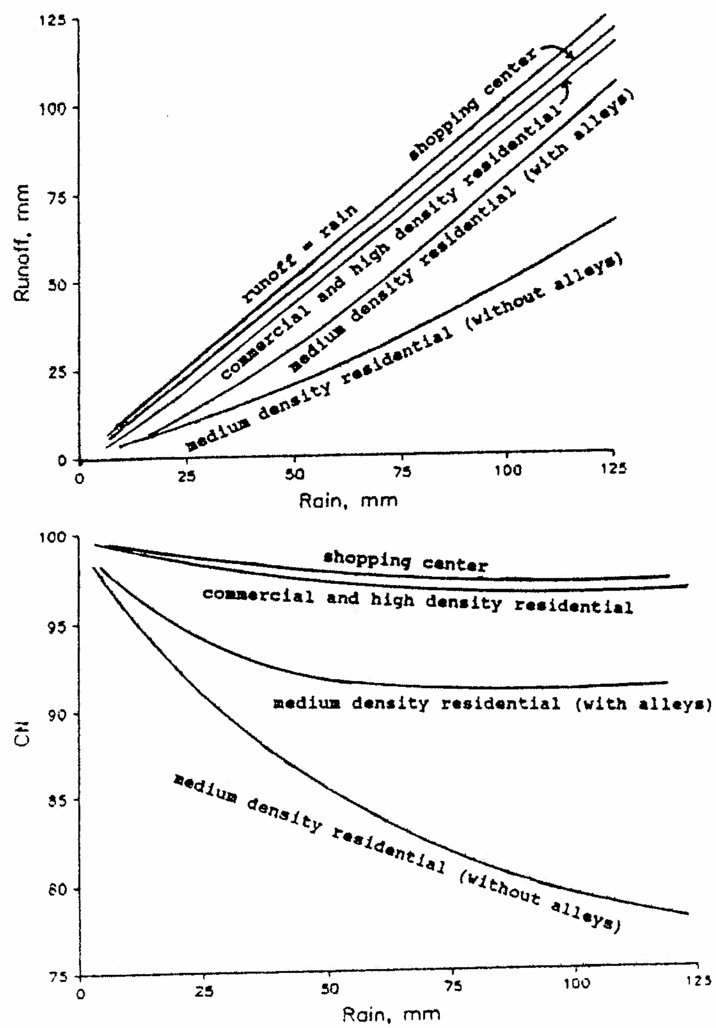


Figure 8.1 Rainfall-Runoff and Curve Numbers for Emery and Thistledowns

Figure 8.2 Rainfall-Runoff and Curve Numbers for Milwaukee NURP Test Watersheds



the data) for the different land uses represented. The observed Toronto outfall data corresponded to a rain depth range of about 0 to 25 mm. The runoff responses for the larger Toronto rains shown on Table 8.2 were therefore extrapolated using the regression equations presented in Section 4 (Table 4.7). The Milwaukee data included rains over this complete range of rains.

The volumetric runoff ratios (Rv) were always low for small rains and increased as the rain depths increased. The more pervious areas changed more slowly and reached lower ultimate Rv values than the mostly impervious areas. In contrast, the curve numbers all started at high values for small rains and decreased as the rain depths increased, with the more impervious areas retaining relatively higher curve numbers over the range of rains. Most of these curve number changes for different rain depths were much greater than the changes in curve numbers for impervious surfaces only, as shown in Section 7.

The lower density residential areas and the industrial area had much lower curve numbers for the large events than recommended by the SCS. The SCS (1986) recommended curve numbers of about 85 for both medium density residential areas in C/D soils (similar to Milwaukee conditions) and industrial areas in B soils (similar to the Toronto conditions). As shown on Table 8.2, these curve numbers may only occur for rains of about 35 mm in the Toronto industrial area and for rains of about 50 mm in the Milwaukee medium density residential area. Larger rains would likely produce much less runoff than predicted using the SCS curve numbers, while smaller rains would produce more runoff than predicted. The differences are most likely due to the Ia/S ratio assumed by the SCS procedure and the specific land surface and drainage characteristics of these watersheds, as discussed previously and in Appendix C.

C. Runoff Responses for Pervious Areas

Runoff responses for pervious areas were needed before the impervious area hydrology model could be applied to complex urban watersheds. The smoothed outfall responses shown in Table 8.2 were developed using monitored events from the eight Milwaukee and the two Toronto watersheds. About 500 events were monitored at the eight Milwaukee locations. A random data subset

of about 100 events were removed from this Milwaukee data set before developing these smoothed Milwaukee curves for final outfall model verification. Pervious area runoff responses were estimated by subtracting the "known" impervious area runoff responses from the smoothed outfall hydrology curves. These pervious area responses were then examined to estimate their relationships to watershed characteristics.

The smoothed outfall responses for the Thistledowns Toronto site and the medium density residential Milwaukee sites (Hastings and Burbank) were mostly influenced by directly connected impervious areas and pervious areas. The appropriate impervious area runoff responses were subtracted from the total outfall response curves to leave pervious area responses for these sites (plus some effect from the partially disconnected impervious areas which were determined simultaneously by examining all of the test watershed data).

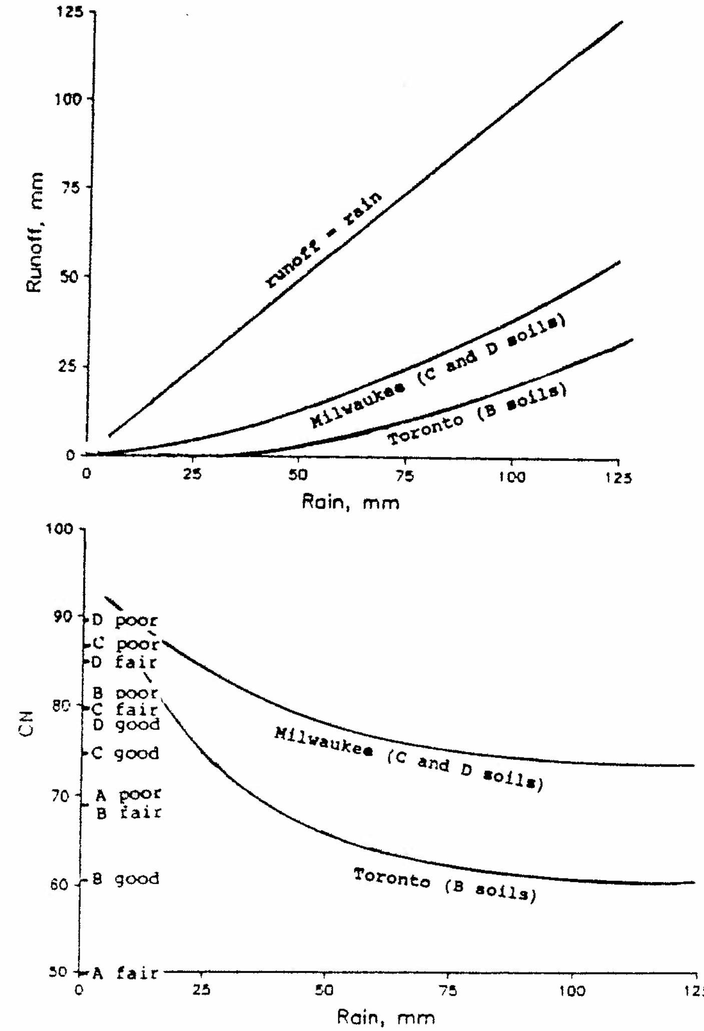
Table 8.3 and Figure 8.3 summarize the estimated pervious area runoff responses for SCS hydrologic soil types B (represented by Toronto) and C and D (represented by Milwaukee). This table also shows the estimated volumetric runoff coefficients and curve

Table 8.3 Pervious Area Runoff Characteristics for Test Watersheds

	Toronto Sites[1]			Milwaukee Sites[1]		
Rain (mm)	Runoff (mm)	Rv	CN	Runoff (mm)	Rv	CN
1	<0.01	<0.01	--	<0.01	<0.01	--
2	<0.01	<0.01	--	<0.01	<0.01	--
3	<0.01	<0.01	--	<0.01	<0.01	--
5	<0.01	<0.01	--	0.48	0.10	96
10	0.10	0.01	86	1.5	0.15	93
15	0.23	0.015	82	2.9	0.19	91
20	0.40	0.02	78	4.0	0.20	89
25	0.63	0.025	75	5.3	0.21	87
30	0.90	0.03	72	6.5	0.22	85
40	1.6	0.04	67	9.2	0.23	81
50	3.5	0.07	66	13	0.26	79
60	6.0	0.10	65	17	0.29	77
70	9.1	0.13	64	22	0.32	76
80	12	0.15	62	26	0.33	74
90	18	0.20	64	32	0.36	74
100	22	0.22	63	39	0.39	73
125	31	0.25	60	56	0.45	73

[1] SCS Hydrologic Soil Types for native, undisturbed, soils:
Toronto: B (mostly loams)
Milwaukee: C/D (mostly sandy clay loams, with some clays)

Figure 8.3 Rainfall-Runoff and Curve Numbers for Pervious Areas in Test Watersheds



numbers for these conditions. Table 8.4 shows the runoff responses for the recommended SCS (1986) curve numbers for pervious areas for A through D soil types for comparison. The curve numbers recommended by SCS are also shown on Figure 8.3. The Toronto and Milwaukee soils appear to have the recommended curve number response for "good" lawns (>75 percent grass cover) after about 75 mm of rain. The estimated curve numbers do not decrease much lower than the SCS recommended values for larger rains, but they are much greater for smaller rains. This would result in much less runoff predicted from pervious areas during small events of most interest in water quality studies and for drainage design studies. Most of the flow (and possibly pollutants) would therefore be incorrectly predicted as originating from impervious areas by using the constant SCS recommended curve number values.

D. Effectiveness of Runoff from Partially Disconnected Impervious Areas

Impervious areas draining to pervious areas (such as paved areas or roofs) obviously lose additional

Table 8.4 Hydrologic Responses for Lawns in Good Condition (>75 percent grass cover) According to SCS Procedures [1]

SCS Soil Hydrologic Group [2]

Rain (mm)	A (CN=39) Runoff (mm)	A Rv	B (CN=61) Runoff (mm)	B Rv	C (CN=74) Runoff (mm)	C Rv	D (CN=80) Runoff (mm)	D Rv
1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
3	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
5	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
10	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
15	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
20	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	0.5	0.03
25	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	2.0	0.08
30	<0.1	<0.1	<0.1	<0.1	2.0	0.07	3.0	0.10
40	<0.1	<0.1	<0.1	<0.1	4.8	0.12	8.1	0.20
50	<0.1	<0.1	2.0	0.04	9.4	0.19	15	0.30
60	<0.1	<0.1	4.1	0.07	14	0.23	20	0.33
70	<0.1	<0.1	6.6	0.09	19	0.27	25	0.36
80	<0.1	<0.1	11	0.13	25	0.32	34	0.43
90	<0.1	<0.1	15	0.17	33	0.36	41	0.46
100	1.0	0.01	20	0.20	51	0.41	50	0.50
125	4.6	0.04	34	0.27	61	0.49	72	0.58

[1] Source: SCS 1986
[2] Assumes constant curve numbers for all rains.

Good: >75% grass cover
Fair: 50 to 75% grass cover
Poor: <50% grass cover

runoff before the flow reaches the drainage system. Most of the impervious area runoff could be lost to infiltration if flowing over long pervious drainage paths, while little additional infiltration would occur if the drainage path was relatively short. The amount of impervious area draining to the pervious drainageway also affects additional infiltration losses. Eight test watersheds represented a variety of conditions, with the partially disconnected impervious areas ranging from about 7.5 to more than 30 percent. The ratio of partially disconnected impervious areas to pervious areas ranged from about 0.35 to 1.3. In addition, some of the areas had relatively short pervious drainage lengths available because of alleys.

The simultaneous evaluation of the outfall runoff residuals for the eight watersheds, after the directly connected impervious runoff responses were removed, allowed both pervious area and partially disconnected impervious area runoff responses to be estimated. The factors of concern were the soil type and the percentage of land cover in the two "unknown" categories. With eight watersheds having very diverse characteristics, sufficient data were available to separate the effects of soil type and land cover.

Table 8.5 shows the estimated drainage effectiveness of partially disconnected impervious areas for various conditions. These values are multipliers to be applied to the appropriate basic impervious area runoff responses shown on Table 7.12. Impervious areas draining across A or B SCS hydrologic soil types, irrespective of land use, or C and D soils for areas having "low density" land uses, behave much like pervious areas themselves. The runoff responses for these partially disconnected impervious areas having these soil and land use conditions should therefore be estimated by using the pervious area runoff responses for the appropriate soil types, as shown on Table 8.3.

The other sets of multipliers on Table 8.5 are for C and D soil types which require either longer flow lengths over pervious areas or relatively small impervious to pervious area ratios for significant infiltration losses. These multipliers are for medium and high density residential and industrial land uses (with or without alleys), and strip commercial and shopping center areas.

Disconnecting paved or roof areas in commercial areas does not result in significant runoff reductions

Table 8.5 Effectiveness of Runoff from "Partially" Disconnected Impervious Areas

	Toronto[1]	Milwaukee Test Areas		
Rain (mm)	Emery and Thistledowns	North Hastings and Burbank[2]	West Congress and Lincoln[3]	State Fair and Wood Center[4]
1	<0.1	<0.1	<0.1	<0.1
2	<0.1	<0.1	0.05	<0.1
3	<0.1	<0.1	0.08	<0.1
5	<0.1	<0.1	0.09	0.47
10	<0.1	<0.1	0.10	0.90
15	<0.1	<0.1	0.17	1.0
20	<0.1	<0.1	0.29	1.0
25	<0.1	<0.1	0.38	1.0
30	<0.1	<0.1	0.46	1.0
40	<0.1	<0.1	0.64	1.0
50	<0.1	<0.1	0.81	1.0
60	<0.1	0.01	0.93	1.0
70	<0.1	0.015	1.0	1.0
80	<0.1	0.02	1.0	1.0
90	<0.1	0.035	1.0	1.0
100	<0.1	0.055	1.0	1.0
125	<0.1	0.085	1.0	1.0

Soil "Type"	B		C	C	C
Partial Discon. Area/	Emery	Thistle			
Pervious Area:	1.3	0.4	0.42	0.35	0.40
Land Use:	light/ medium indus.	resid/ comm.	medium density residential	medium density residential with alleys	Strip comm. and high density residential with alleys

[1] Use pervious area runoff relationships for all disconnected impervious surfaces for all land uses located in areas having "A" and "B" soils, and for low density residential, institutional (low building density) and open space (parks, golf courses, etc.) areas having "C" and "D" soils.

[2] Use these relationships for "C" and "D" soils for medium and high density residential, industrial and high building density institutional land uses (without alleys). Roof drains located "close" (less than about two meters) to connected pavement are considered directly connected.

[3] Use these relationships for "C" and "D" soils for medium and high density residential, industrial and high building density institutional land uses with alleys.

[4] Use these relationships for "C" and "D" soils for strip commercial and shopping center land uses.

for rains greater than about 5 or 10 mm. In contrast, disconnecting paved or roof areas in medium and high density residential or industrial areas without alleys can result in substantial runoff reductions. However, if alleys occur in these areas, then the runoff reduction benefits are greatly reduced because of the much shorter flow lengths over the available pervious areas.

Example calculations showing how to use Tables 8.5 and 7.12 to estimate the runoff responses of partially disconnected impervious areas follow. The runoff effectiveness multiplier for a medium density residential area having C or D soils and alleys is 0.38 for a rain depth of 25 mm (from Table 8.5). The basic volumetric runoff coefficient for flat roofs for this rain is 0.84 (from Table 7.12). Therefore, if the downspout from a flat roof is disconnected and its runoff flows over typical soils in this area before reaching the drainage system, the resultant flat roof volumetric runoff coefficient is 0.38 times 0.84, or 0.32. This results in a roof runoff volume reduction of about 60 percent compared to directly connecting the roof downspouts to the drainage system. The following list shows other examples of resultant runoff responses for partially disconnected flat roofs for a variety of rains, land uses, and soil types.

Volumetric Runoff Coefficients for:

Rain (mm)	A or B all uses	C or D indus. (no alleys)	C or D indus. (alleys)	C or D commer.
3	<0.01	<0.1	0.02	<0.03
10	0.01	<0.07	0.07	0.65
25	0.025	<0.08	0.32	0.84
50	0.07	<0.09	0.73	0.90
90	0.20	0.03	0.94	0.94

Except for most of the commercial runoff responses, and for the largest rains for the industrial area with alleys, these runoff responses are much less than shown previously on Table 7.12 for directly connected large flat roofs.

The current version of the SCS curve number procedure (SCS 1986) contains a method to adjust outfall curve numbers for disconnected impervious areas. This SCS procedure uses the percentage of the

area that is impervious and the pervious area curve number. The above industrial area example with the Emery area characteristics was used with this SCS procedure for comparison. For A or B soils, the partially disconnected impervious surfaces did not contribute any runoff for the above rains according to the SCS procedures and as estimated in Table 8.5. For C soils, the partially disconnected impervious areas contributed only about 6 percent of the impervious area contributions for the 90 mm rain and no contributions for smaller rains according to the SCS procedures, and again in general agreement with the multipliers shown on Table 8.5 for similar conditions. These methods therefore result in similar partially disconnected impervious area responses for these conditions. However, the SCS procedure does not consider the presence of alleys or the effects of drainage density (such as in commercial areas) in limiting the benefits of impervious areas draining to pervious areas. As an example, the SCS procedure would substantially over-estimate runoff from partially disconnected impervious areas in medium or high density residential, industrial, and some institutional land uses having alleys.

E. Verification of Homogeneous Area Hydrology Models

The impervious area runoff responses summarized in Table 7.12 were used in conjunction with the pervious area and partially disconnected impervious area runoff response estimates presented in Table 8.5 to predict outfall runoff for all of the monitored Toronto events and the 100 random events monitored in Milwaukee. The Milwaukee events represented a wide range of monitored rains that allowed the impervious area hydrology model to be verified for a broad range of rain and watershed conditions. These 100 Milwaukee events were not used in formulating the homogeneous area models and were therefore an independent verification test of the complete model.

Figures 8.4 through 8.7 illustrate the behavior of the residuals (observed runoff volumes minus predicted runoff volumes) for this runoff model verification analysis. The Toronto residual analysis showed predicted runoff volumes within about 2 mm of observed runoff volumes for almost all events, except for about three large events (out of about 85). The Milwaukee

Figure 8.4 Observed and Predicted Runoff for Emery

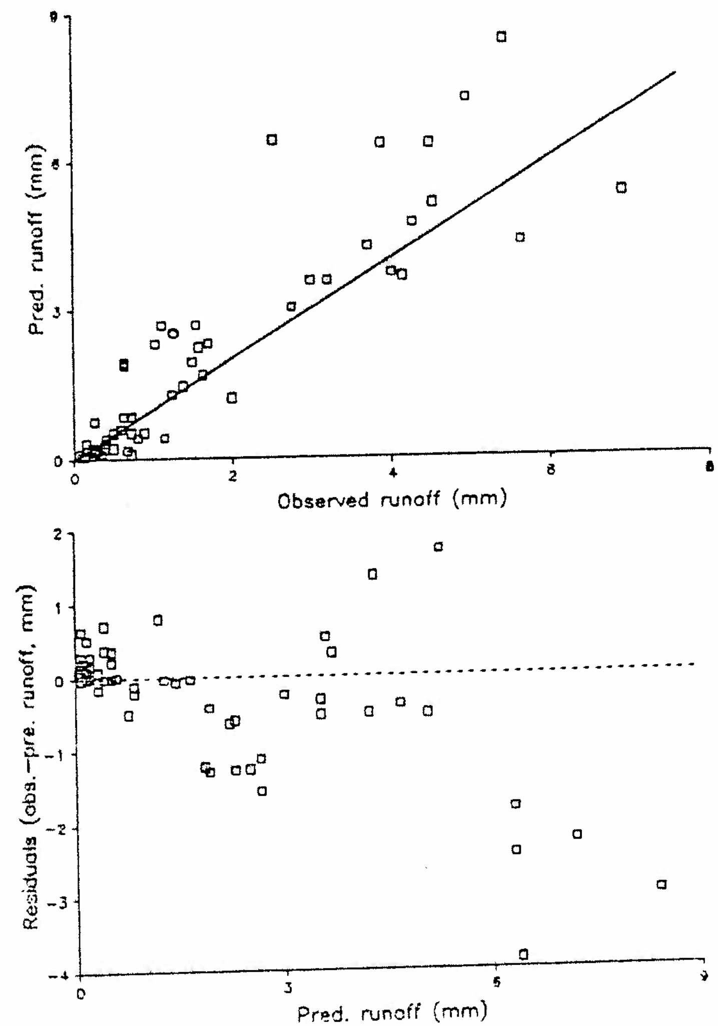


Figure 8.5 Observed and Predicted Runoff for Thistledowne

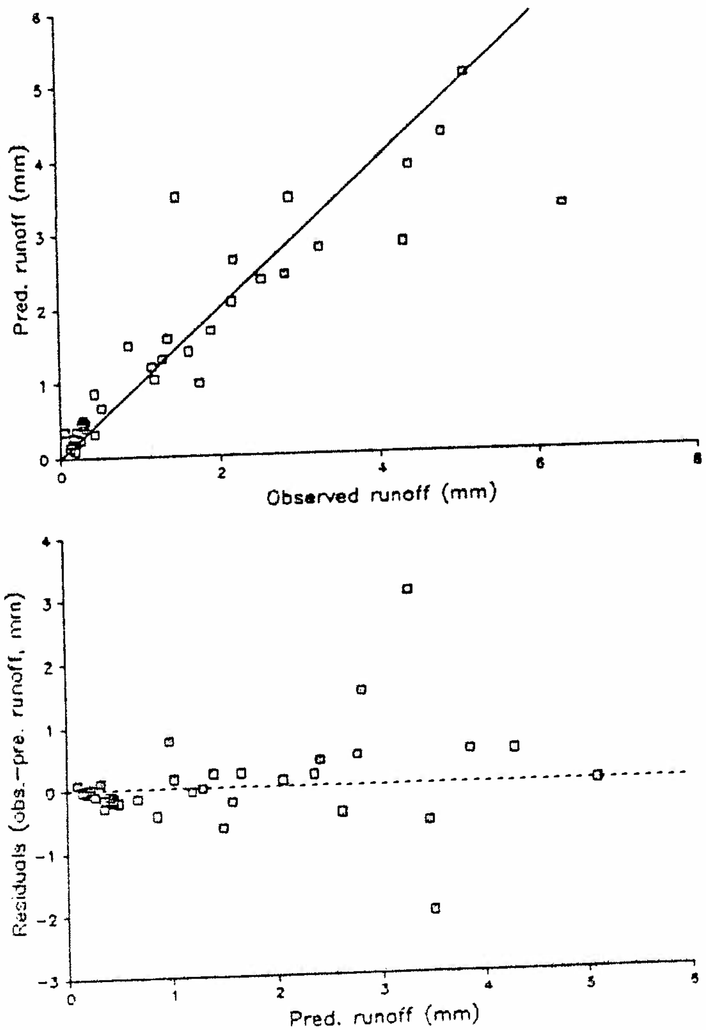


Figure 8.6 Observed and Predicted Runoff for Random Milwaukee NURP Data

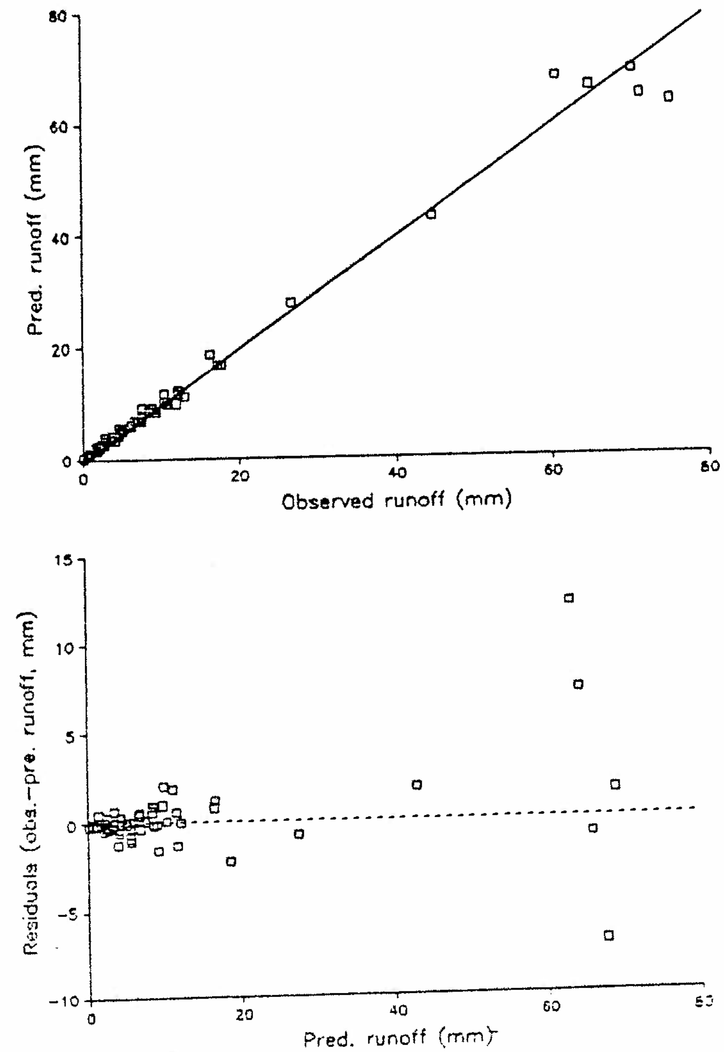
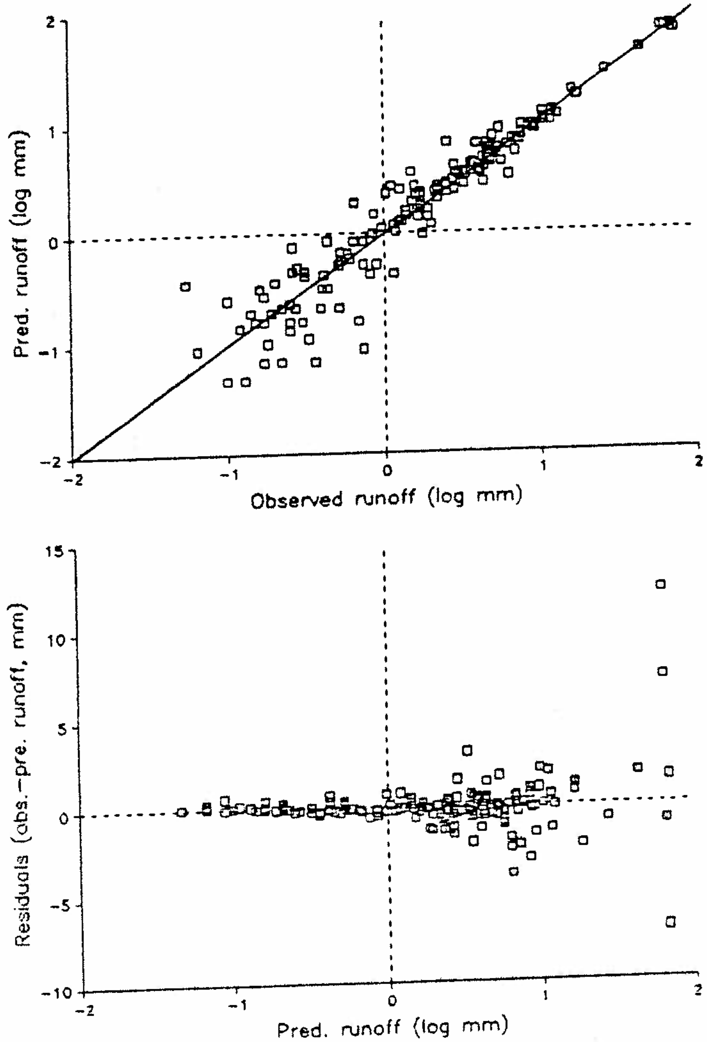


Figure 8.7 Observed and Predicted Runoff for All Toronto and Random Milwaukee NURP Data



sites' residuals also indicated good fits of the models, even over a wider range of runoff conditions. Only about three Milwaukee events (out of 100) had runoff prediction residuals greater than 2 mm. The largest Milwaukee residuals were also observed for the largest monitored events.

F. Use of the Homogeneous Area Hydrology Models in Complex Watersheds

Urban Runoff Flow Sources

One of the major objectives of this dissertation was to demonstrate how sources of urban runoff flows and pollutants could be determined for a variety of precipitation and land use conditions. This subsection illustrates how the homogeneous area runoff models, integrated into the Source Loading and Management Model (SLAMM), can be used to estimate runoff source areas. Section 10 shows examples of SLAMM being used to estimate sources of urban runoff pollutants, and the effects of different urban runoff controls on the source contributions of flows and pollutants.

Table 8.6 summarizes the estimated annual mass balances for water at the two Toronto test watersheds, based on almost complete monitoring of all flows. Baseflows contributed substantial portions of the annual urban runoff flow volumes. Warm weather baseflows occurred from May through October and averaged about 0.25 L/sec-ha at the Emery industrial site and 0.30 L/sec-ha at the Thistledowns mixed residential and commercial site. The warm weather baseflows ranged from about 0.01 to 1.5 L/sec-ha. The cold weather baseflows were less, averaging about 0.12 L/sec-ha at Emery and 0.22 L/sec-ha at Thistledowns. The cold weather baseflows ranged from about 0.03 to 0.57 L/sec-ha.

The runoff during warm weather rains only accounted for 17 to 30 percent of the total annual urban runoff water yields. The rain and snowmelt induced runoff combined accounted only for about 50 percent of the annual urban runoff discharges at these two watersheds. Baseflows accounted for about 50 percent of the annual flows at each location and, depending on control program objectives, could have significant effects on attaining the desired benefits from any management program. Pollutant characteristics

Table 8.6 Annual Urban Runoff Volumes by Time Period

	Emery		Thistledowns	
	m^3/ha	% of annual	m^3/ha	% of annual
Warm Season [1]				
Baseflow	2100	41.2%	1700	30.7%
Stormwater	1500	29.5	950	17.1
Total warm season	3600	70.7	2650	47.8
Cold season [2]				
Baseflow	660	13.0	1100	19.8
Snowmelt	830	16.3	1800	32.4
Total cold season	1490	29.3	2900	52.2
Annual Total	5190	-	5550	-

[1] March 15 through December 15
[2] December 15 through March 15

of these flows can also have dramatic effects on the relative pollutant contributions of each flow component which also must be considered when designing an urban runoff management program.

The homogeneous area runoff responses presented earlier have been used in SLAMM to estimate the relative source area flow contributions during warm weather rains for a range of rain depths. Figures 8.8 and 8.9 show how differently the Emery and Thistledowns sites responded for a range of rains. The industrial site had a much more even runoff response during all rains, with connected roofs contributing most of the flow (about 55 percent) and paved parking and storage areas contributing about 35 percent of the flow. Streets made up most of the remainder of the flow contributions. If reducing runoff volumes (and/or flow rates) from this industrial area was of concern, disconnecting the rooftops and the parking areas could have dramatic benefits.

The mixed residential and commercial site had a more complex flow structure, with streets and connected roofs each contributing about 35 to 45 percent of the flows for small rains (or at the beginning of large rains), with reduced contributions (down to about 10 to

Figure 8.8 Emery Urban Runoff Flow Sources

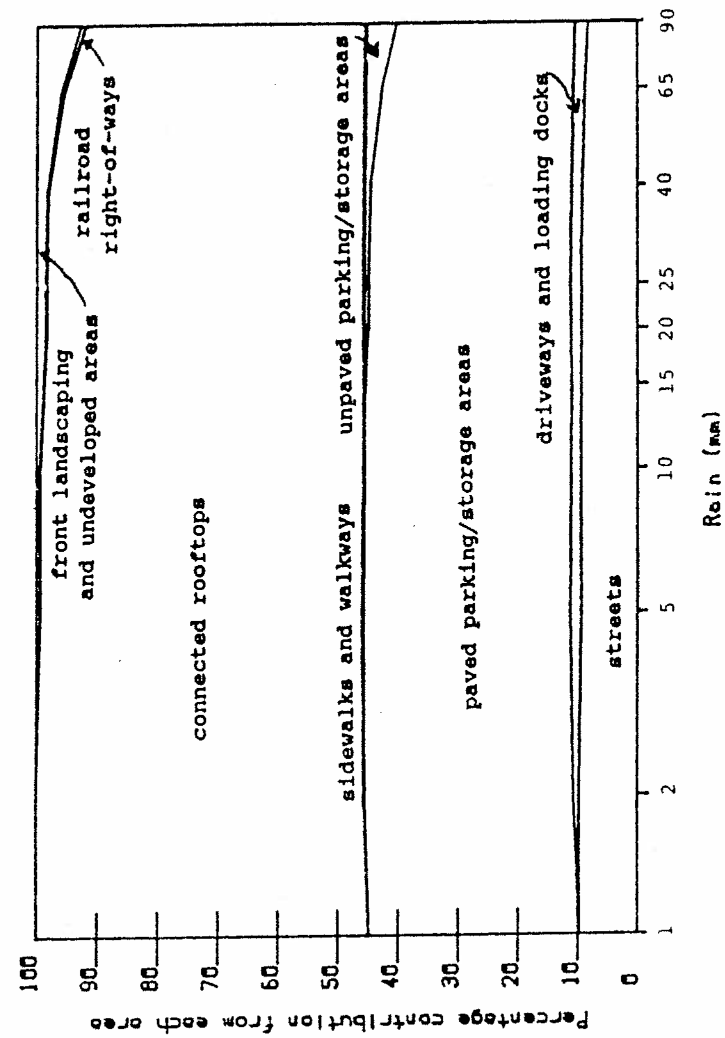
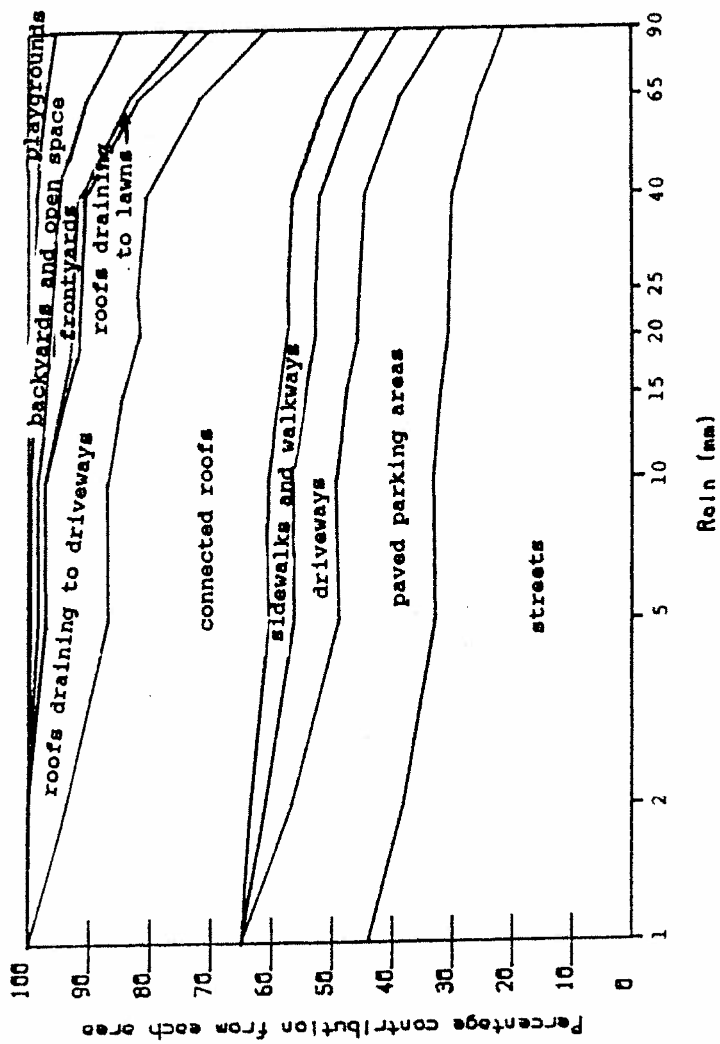


Figure 8.9 Thistledowns Urban Runoff Flow Sources



20 percent each) during the largest rains. Generally, directly connected paved areas contributed most of the runoff during small rains, with more important contributions from pervious areas and partially disconnected impervious areas as the rain depths increased. A reasonable runoff volume control program at Thistledowns may be limited to disconnecting the directly connected roof drains. Installing many infiltration devices in small paved areas, or removing curbs and gutters and installing grass drainage swales is usually not very feasible as a retro-fitting contro program.

The source area contributions during snowmelt events are assumed to be directly related to the sourc areas. During initial periods of snowmelt, or for smal afternoon snowmelts, areas adjacent to the drainage system (such as street-side snow windrows) probably contributed more melt water (and pollutants) than area farther from the drainage system. During major snow melting periods, roofs, frontyards, and backyards in residential areas each contributed approximately 20 to 30 percent of the total runoff volume. Paved parking and storage areas, roofs, landscaped areas, and open spaces in industrial areas each contributed approximately 20 to 30 percent of the total runoff volume.

The relative importance of the different sources of baseflows are more difficult to estimate than source areas during rain runoff events. Groundwater or domestic water infiltration may be the most significant contributor of baseflow volumes. Other source contributions are mostly related to individual behavior of residents within the areas. It only requires a small number of "backyard mechanics" who dump their used oil into the storm drainage inlets or flush their radiators on their driveways to significantly affect the baseflow pollutant discharges. Similarly, if excessive irrigation of lawns is common for an area, increased baseflow volumes would occur.

Use of Model to Calculate Curve Numbers Reflecting Specific Development Practices, Urban Runoff Controls, and Different Rain Depths

Another important objective of this dissertation was to develop a method that could be used to integrate the water volume reduction benefits of water quality control devices into storm drainage design practice. SLAMM contains the calibrated homogeneous area

hydrologic models previously discussed and produces appropriate curve numbers for specific development practices, water quality controls, and different rain depths. Table 8.7 is an example of how SLAMM was used to produce different curve numbers for an industrial area, depending on the extent of disconnections of the impervious areas and if grass swales were used for roadside drainage.

Conventional development practices and the use of curbs and gutters would require the use of curve numbers ranging from about 88 for the 100-year storm to 92 for the 2-year storm. The use of grass swales could totally eliminate all runoff for rains of less than the 2-year frequency, and produce significant volume reductions even for the 24-hr, 100-year storm (reducing the CN from 88 to 70, and the Rv from 0.76 to 0.45, for a percentage flow volume reduction of about 40 percent). Retro-fitting other controls in an industrial area with existing curbs and gutters could still produce significant benefits; disconnecting about one-half of the connected roofs and parking areas would result in about a 20 percent reduction in the 100-year storm flow volume.

Table 8.7 Calculated Runoff Responses for 24-hour Storms for Light Industrial Areas with Different Water Quality Controls

Development Conditions:		2yr 58mm (2.3in)	5-yr 86mm (3.4in)	10-yr 100mm (3.9in)	25-yr 112mm (4.4in)	50-yr 127mm (5.0in)	100-yr 142mm (5.6in)	Flow Reductions for typical 1-yr. events (compared to typical dev. with curb & gutters)
		Approximate Storm Return Frequency and Rain Volume						
Typical development with curbs and gutters	RV:	0.65	0.74	0.75	0.76	0.76	0.76	0%
	CN:	92	92	91	90	89	88	
Typical development with grass swales (64mm/hr) infiltration	RV:	0.0	0.23	0.31	0.37	0.41	0.45	100%
	CN:	–	67	69	70	70	70	
Extensive disconnections of roofs and parking areas with curbs and gutters	RV:	0.43	0.53	0.57	0.58	0.59	0.60	35%
	CN:	85	83	82	82	82	80	
Extensive disconnections of roofs and parking areas with grass swales (64mm/hr infiltration)	RV:	0.0	0.12	0.21	0.23	0.28	0.32	100%
	CN:	–	<60	60	60	62	62	

G. Summary of Impervious Area Hydrology Model Verification and Model Use in Complex Urban Watersheds

This section developed pervious and partially disconnected impervious area runoff responses by subtracting the known directly connected impervious area contributions from smoothed outfall runoff curves for eight very different watersheds. Independent Milwaukee test watershed data and all of the Toronto test watershed data for individual events were then compared to the predicted outfall runoff expected from using these homogeneous source area runoff models. Predicted outfall runoff was within about 2 mm of the observed runoff responses for more than 90 percent of the verification events tested.

Before these verified urban hydrology models could be used in complex watersheds, it was necessary to examine the warm and cold weather baseflow and snowmelt data obtained during this research. The annual mass balance of runoff flows indicated that rain induced runoff only contributed about 20 to 30 percent of the annual urban runoff discharges, with the remaining urban runoff flows originating from baseflows and snowmelts. Commonly neglected baseflows contributed about one-half of the total annual urban runoff flows at both test watersheds.

Example uses of these hydrology models included estimating source area contributions and curve numbers for different watershed development practices and water quality controls. Source area runoff contribution information is needed to evaluate source area flow reduction controls. Reduced runoff volumes, due to specific development practices or runoff controls, are usually overlooked and can have significant effects on curve numbers used in designing storm drainage systems.

The next section of this dissertation discusses particulate washoff tests that were used to develop a general particulate washoff model. The hydrology information presented in the previous sections of this dissertation is used in SLAMM, along with source area pollutant information, to estimate source area and seasonal mass balances for selected pollutants in Section 10. These mass balances are needed to design and evaluate urban runoff flow and pollutant management programs.

SECTION 9

STREET DIRT WASHOFF

A. Introduction

This section summarizes the street dirt washoff tests conducted in the test watersheds. The objectives of these washoff tests were to identify the significant factors affecting street dirt washoff and to develop prediction equations sensitive to these significant factors. It was also desired to investigate the effects of particulate "availability", especially for different particle sizes and rain conditions, on these equations.

The report prepared for the Ontario Ministry of the Environment (Pitt and McLean 1986) contains the complete data. Non-linear model parameters were fitted to these washoff data, reflecting significant environmental factors.

The washoff tests found important variations in washoff for different sized particles and demonstrated the importance of not confusing "total" and "available" initial street dirt loadings, as has been common in previous model applications. Relative pollutant strengths (mg pollutant/kg total residue) were simultaneously measured for the different source areas (also described in Pitt and McLean 1986) and were used with this accumulation and washoff information for incorporation into the Source Loading and Management Model (SLAMM). The calibrated model was used for each outfall event that was monitored to predict warm weather flow-weighted concentrations for comparisons with the outfall measurements for verification, as reported in the next section.

B. Background

Ellis et al. (1981) examined heavy metal discharges in urban runoff and concluded that the degradation of the road surface and traffic related discharges are responsible for most of the particulate discharges in urban runoff. He also found that the smallest particulates from urban areas are usually discharged during the early parts of storms, but small particulates from impervious surfaces may also be discharged during later parts of storms. Shaheen (1975)

found that road surface particulates and polluted area soils (affected by traffic related pollutants) contribute most of the urban runoff particulate pollutants. As noted earlier, many urban runoff models assume that "all" of the pollutants and runoff flows in urban areas originate from directly connected impervious areas. Section 8 showed how much of the runoff volume from the test watersheds originated from impervious surfaces during most rains. The correct interpretation of particulate washoff from impervious surfaces is therefore critical to understanding urban runoff quality. This subsection summarizes some of the procedures that are commonly used to estimate particulate washoff from impervious surfaces, presents the results of the washoff tests conducted during this research, and develops the resulting revised washoff model.

Washoff of particulates from impervious surfaces is dependent on the available supply of particulates and the capacity of the runoff to transport the loosened material (Ammon 1979). The accumulation of the material is dependent on many site specific land use and geographic features, plus the intended or unintended losses of materials, as described earlier in Figure 3.2.

Brief descriptions follow of two methods (the Yalin equation and the Sartor and Boyd equation) currently used in most urban runoff studies for estimating particulate washoff from impervious surfaces. They can be used to obtain satisfactory estimates of particulate washoff, if their limitations are recognized and if rough estimates are all that are required. Unfortunately, they are often used in situations beyond their limits (such as for small rains, unusual street dirt loadings, or rough pavement textures). Certain washoff equation parameters have also been misunderstood (such as confusing total dirt load with "available" dirt load). The use of these washoff equations in large and well documented urban runoff computer models also implies more confidence in their accuracy than may be warranted.

A recent study is briefly summarized that found significant washoff differences for various particle sizes. These observed washoff quantities are compared to the values obtained with these two washoff models, but the observed washoff quantities are shown to be much less than predicted with the washoff equations.

These data observations and the existing washoff models' inabilities to accurately predict washoff lead to the series of washoff tests conducted during this research and the development of washoff models sensitive to important environmental conditions.

Yalin Equation

Novotny and Chesters (1981) presented the Yalin equation as the best candidate from the many models presented in the literature to describe sediment washoff and transport in urban areas. The Yalin equation relates the sediment carrying capacity to runoff flow rate (Yalin 1963). Yalin assumed that sediment motion begins when the lift force of flow exceeds a critical lift force. Once a particle is lifted from the bed, the drag force of the flow moves it downstream until the weight of the particle forces it back to the bed. The Yalin equation is used to predict particle transport, for specific particle sizes, on a weight per unit flow width basis. It is used for fully turbulent channel flow conditions, typical of shallow overland flow in urban areas. The receding limb (tail) of a hydrograph may have laminar flow conditions, and the suspended sediment carried in the previously turbulent flows would settle out. The predicted constant Yalin sediment load would therefore only occur during periods of rain; and the sediment load would decrease, due to sedimentation, after the rain stops. The equation is presented in the following form:

$$p = 0.635\ s\ [1 - (1 / a\ s)\ \ln\ (1 + a\ s)]$$

where p = particle transport, grams/meter-second and a and s are calculated, based on particle density, particle diameter, and shear velocity.

To use the equation, the particle shear velocity (v_*, m/sec) must be calculated:

$$v_* = (gHS)^{1/2}$$

where g = acceleration of gravity = 9.81 m/sec^2
H = flow depth, meters
S = energy gradient slope, m/m

The particle Reynolds number (X) must also be known:

$$X = v_*\ D / u$$

where D = particle diameter, meters
u = kinematic viscosity of fluid = 10^{-6} m^2/sec for water

The critical particle bedload tractive force (Y_{cr}), the tractive force at which the particle begins to move, can be obtained from a Shield's diagram (Figure 9.1). Shen (1981) warned that Shield's diagram cannot be used alone to predict "self-cleaning" velocities, it gives only a lower limit below which deposition will occur. It defines the boundary between bed movement and stationary bed conditions. The diagram does not consider the particulate supply rate in relationship to the particulate transport rate. Reduced particulate transport occurs if the sediment supply rate is less than the transport rate.

The actual tractive force is also calculated:

$$Y = v_*^2 / (p_s - 1) g * D$$

where p_s = specific density of particle, g/cm^3

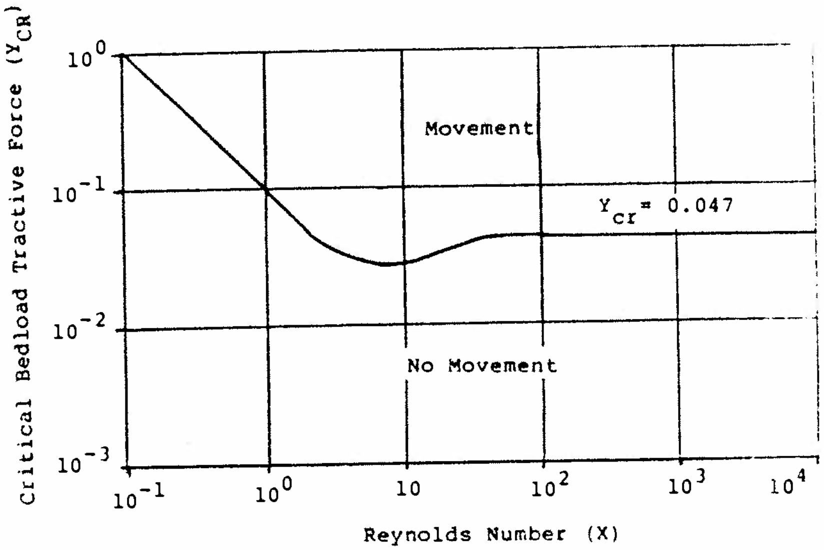


Figure 9.1 Shields' Diagram for Particle Tractive Force

Source: **Novotny and Chesters 1981**

The Yalin coefficients can be calculated knowing Y, Y_{cr}, and p_s:

$$s = Y / Y_{cr}$$

$$\text{and } a = 2.45\ p_s^{-0.4}\ (Y_{cr})^{1/2}$$

The Yalin equation by itself is therefore not sensitive to particulate supply; it only predicts the carrying capacity of flowing waters. Models must be used that account for total particulate discharge and "stop" transport when the particulate supply is exhausted.

Besides the particulate supply rate, the Yalin equation is also very sensitive to local flow parameters (specifically gutter flow depth); a hydraulic model that can accurately predict sheetflow across impervious surfaces and gutter flow is needed. Sutherland and McCuen (1978) statistically analyzed a modified form of the Yalin equation, in conjunction with a hydraulic model (the Basic Inlet Hydrograph Model - BIHM - developed by Ragan and Root 1974), for different gutter flow conditions. Except for the largest particle sizes, the effect of rain intensity or particle washoff was negligible. A set of equations, shown on Table 9.1, were developed relating the percentage washoff (TS_i) of each of six particle sizes to gutter slope, impervious area, initial solids loading, and the gutter length before the storm drain inlet. These washoff percentages assume a one-hour uniform rain of 13 mm. These washoff percentages can then be modified for other total rains, by the K_j factors given in Table 9.2:

$$TS_j = K_j \; TS_i$$

where TS_j = percent total solids removal (for a specific size range)

TS_i = percent total solids removal for the standard 13 mm rain (for a specific size range)

K_j = factor relating the standard rain to the actual rain

The Yalin equation is based on classical sediment transport equations, and requires some assumptions concerning the micro-scale aspects of gutter flows and street dirt distributions. The Yalin equation, as typically used in urban runoff models, assumes that all

Table 9.1 Total Solids Removed by Rainfall (uniform rainfall of one-half inch over one-hour duration)

Range	R	S_e(%)	Equation
1	0.92	2.29	$T_1 = 91.4+0.76\ G^{1.92}+0.1\ I-0.01\ S-0.0032\ L$
2	0.89	1.49	$T_2 = 95.6+0.65\ G^{1.55}+0.061\ I-0.0058\ S-0.0028\ L$
3	0.94	3.74	$T_3 = 83.6+G^{2.14}+0.2\ I-0.0019\ S-0.0063\ L$
4	0.95	6.60	$T_4 = 64.2+1.35\ G^{2.45}+0.39\ I-0.036\ S-0.0073\ L$
5	0.96	9.59	$T_5 = 33.6\ 1.58\ G^{2.7}+I^{0.9}-0.062\ S$
6	0.99	3.40	$T_6 = (G-1.44)(-3.7+0.5\ I^{0.95}-0.02\ S+0.026\ L)$

R = correlation coefficient

S_e = standard error of estimate.

T_i = removal percentage for particle size range i

G = slope of gutter in percent.

I = impervious area in percent.

S = initial total solids loading in lbs/curb mile.

L = length of gutter in feet.

Note: if T_i exceeds 100 it is assumed to equal 100 in the model.

Source: Sutherland and McCuen, 1978

Table 9.2 K_j Values to be used in the Equation $TS = K_j (TS_i)$

TS_i	Total Volume j: 1/8	1/4	1/2	3/4	1	1-1/4	1-1/2	1-3/4	2	2-1/2	3
100	0.84	0.92	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
97	0.835	0.938	1.0	1.021	1.031	1.031	1.031	1.031	1.031	1.031	1.031
94	0.808	0.915	1.0	1.032	1.053	1.064	1.064	1.064	1.064	1.0064	1.064
93	0.731	0.882	1.0	1.048	1.075	1.075	1.075	1.075	1.075	1.075	1.075
90	0.778	0.889	1.0	1.055	1.089	1.1	1.111	1.111	1.111	1.111	1.111
88	0.670	0.852	1.0	1.063	1.091	1.108	1.119	1.131	1.136	1.136	1.136
84	0.417	0.798	1.0	1.083	1.119	1.143	1.161	1.178	1.190	1.190	1.190
76	0.303	0.658	1.0	1.125	1.184	1.224	1.25	1.263	1.270	1.289	1.316
72	0.239	0.542	1.0	1.125	1.208	1.264	1.305	1.333	1.347	1.369	1.389
64	0.156	0.375	1.0	1.219	1.344	1.406	1.469	1.515	1.563	1.563	1.563
61	0.082	0.295	1.0	1.230	1.352	1.426	1.492	1.533	1.582	1.606	1.639
45	0.044	0.178	1.0	1.489	1.689	1.811	1.911	1.978	2.044	2.149	2.222
44	0.057	0.159	1.0	1.477	1.704	1.841	1.954	2.045	2.114	2.182	2.232
15	0.0	0.133	1.0	2.6	3.933	4.733	5.233	5.6	5.9	6.233	6.333
2	0.0	0.0	1.0	4.0	11.0	20.0	26.5	30.5	33.75	38.0	41.0

TS_j = the percentage removal of total solids in a particle size range due to a total rainfall volume measured in inches.

TS_i = the percentage removal of total solids in a particle size range due to a total rainfall volume of 1/2 inches.

Source: Sutherland and McCuen, 1978

particles lie within the gutter, and no significant washoff occurs by sheetflows traveling across the street towards the gutter. The early measurements of across-the-street dirt distributions made by Sartor and Boyd (1972) indicated that about 90 percent of the street dirt was within about 30 cm of the curb face (typically within the gutter area). These measurements, however, were made in areas of no parking (near fire hydrants because of the need for water for the sampling procedures that were used), and the traffic turbulence was capable of blowing most of the street dirt against the curb barrier (or over the curb onto adjacent sidewalks or landscaped areas). In later tests, Pitt (1979) examined street dirt distributions across-the-street in many situations. He found distributions similar to Sartor and Boyd's observations only on smooth streets, with moderate to heavy traffic, and with no on-street parking. In many cases, most of the street dirt was actually in the driving lanes, trapped by the texture of rough streets. If on-street parking was common, much of the street dirt was found on the outside edge of the parking lanes, where the resuspended (in air) street dirt blew against the parked cars and settled to the pavement. Some later modeling efforts (most notably later versions of the MUNP and PTM models, Sutherland personal communication) adjusted the total street loading to estimate the loading present only in the gutter. Washoff of in-street particulates was still not considered.

Another process that may result in washoff less than predicted by Yalin is bed armoring (Sutherland et al. 1982?). As the smaller particulates are removed, the surface is covered by predominantly larger particulates which are not effectively washed off by the rain. Eventually, these larger particulates hinder the washoff of the trapped, under-lying, smaller particulates. Debris on the street, especially leaves, can also effectively armor the particulates, reducing the washoff of particulates to very low levels (Singer and Blackard 1978).

Sartor and Boyd Washoff Equation

Observations of particulate washoff during controlled tests may result in empirical washoff models that are not as limited as incomplete theoretical models. Washoff experiments using actual streets and natural street dirt and debris are affected by street dirt distributions and armoring. Their disadvantage is

the assumption of transferability. If the washoff experiments are conducted for many situations then it may be possible to use the resultant model for other situations.

The earliest controlled street dirt washoff experiments were conducted by Sartor and Boyd (1972) during the summer of 1970 in Bakersfield, California. Their data was used in many urban runoff models (including SWMM, Huber and Heaney 1981; STORM, COE 1975; and HSPF, Donigian and Crawford 1976) to estimate the percentage of the available particulates on the streets that would wash off during rains of different magnitudes. They used a rain simulator having many nozzles and a drop height of 1-1/2 to 2 meters in street test areas of about 5 by 10 meters. Tests were conducted on concrete, new asphalt, and old asphalt, using simulated rain intensities of about 5 and 20 mm/hr. They collected and analyzed runoff samples every 15 minutes for about two hours for each test. Figure 9.2 shows two plots of their data, showing the asymptotic shape of the accumulative washoff curves for several particle sizes. Sartor and Boyd fitted their data to an exponential curve, assuming that the rate of

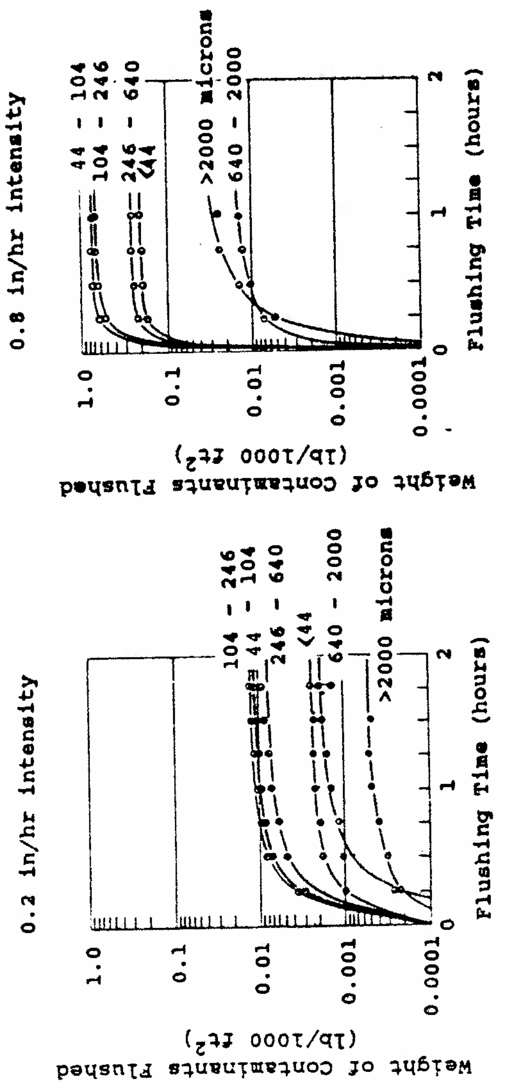


Figure 9.2 Sartor and Boyd Street Dirt Washoff Data (new asphalt)

Source: Sartor and Boyd 1972

particle removal of a given size is proportional to the street dirt loading and the constant rain intensity:

$$dN/dt = k\ r\ N$$

where dN/dt = the change in street dirt loading per unit time

k = proportionality constant

r = rain intensity (in/hr)

N = street dirt loading (lb/curb-mile)

This equation, upon integration, becomes:

$$N = N_o\ e^{-krt}$$

where N = residual street dirt load (after the rain)

N_o = initial street dirt load

t = rain duration

Street dirt washoff is therefore equal to N_o minus N. The variable combination rt, or rain intensity times rain duration, is equal to total rain volume (R). This equation further reduces to:

$$N = N_o e^{-kR}$$

Therefore, this equation is only sensitive to total rain, and not rain intensity.

Because of decreasing particulate supplies, the exponential washoff curve also predicts decreasing concentrations of particulates with time since the start of a constant rain (Alley 1980 and 1981).

The proportionality constant, k, was found by Sartor and Boyd to be slightly dependent on street texture and condition, but was independent of rain intensity and particle size. The value of this constant is usually taken as 0.18/mm, assuming that 90 percent of the particulates will be washed from a paved surface in 1 hour during a 13 mm/hour rain. However, Alley (1981) fitted this model to watershed outfall runoff data and found that the constant varied for different storms and pollutants, for a single study area. Novotny (undated) examined "before" and "after" rain event street particulate loading data using the Milwaukee NURP data and found almost a three-fold difference between the constant value for fine (<45 microns) and medium sized particles (100 to 250 microns); 0.026/mm

for the fine particles and 0.01/mm for the medium sized particles, both much less than the "accepted" value. Jewell et al. (1980) also found large variations in outfall "fitted" constant values for different rains compared to the typical default value. Either the assumption of the high removal of particulates during the 13 mm/hr storm was incorrect or/and the equation cannot be fitted to outfall data (which assumes that all the particulates are originating from homogeneous paved surfaces during all storm conditions).

This washoff equation has been used in many urban runoff models (including SWMM, STORM, and HSPF), but the N_o factor has been frequently misinterpreted. It has been assumed to be the total initial street loading, when in fact it is only the portion of the total street load available for washoff. STORM and SWMM use an availability factor (A) for particulate residue as a calibration procedure in order to reduce the washoff quantity for different rain intensities (Novotny and Chesters 1981):

$$A = 0.057 + 0.04 (r^{1.1})$$

where r is the rain intensity (mm/hr), and A must be less than 1.0. This regression equation is used to adjust the relative importance of the particulate residue contributions from pervious and impervious source areas. This availability factor is equal to 1.0 for all rain intensities greater than about 18 mm/hr. For rains of 1 mm/hr, this availability factor reduces to about 0.10. HSPF does not use an availability factor in an attempt to be "more universally applicable" (Donigian and Crawford 1976). Instead, calibration of observed with predicted outfall yields are used to "adjust" the accumulation and washoff rates directly in HSPF. Ammon (1979) found that the availability factor in SWMM does not really have a significant effect on the variation of the predicted runoff load. However, it does affect the relationship between the runoff volume and the particulate washoff (and therefore concentration).

Jewell et al. (1978) stressed the need to have local calibration data before using the exponential washoff equation, as the default values can be very misleading. Ammon (1979) concluded that the exponential washoff equation for impervious areas is justified, but

washoff coefficients for each pollutant would improve its accuracy.

<u>Recent Street Dirt Washoff Observations and Comparisons with the Yalin and Sartor and Boyd Washoff Equations</u>

The Bellevue, Washington, urban runoff project (Pitt 1984) included about 50 pairs of street dirt loading observations close to the beginnings and ends of rains. These before and after loading values were compared to determine significant differences in loadings that may have been caused by the rains. The observations were affected by rains falling directly on the streets, along with flows and particulates originating from non-street areas. The net loading differences were therefore affected by street dirt washoff (by direct rains on the street surfaces and by gutter flows augmented by "upstream" area runoff) and by erosion products that originated from non-street areas that may have settled out in the gutters. When all the data were considered together, the net loading difference was about 10 to 13 grams/curb-m removed. This amounted to a street dirt load reduction of about 15 percent, which was much less than predicted using the previously described washoff models.

Very large reductions in street dirt loadings for the small particles were observed in Bellevue, but the largest particles actually increased in loadings (due to settled erosion materials). The particles were not source limited, but armor shielding may have been important. Most of the weight of solid material in the runoff was in the fine particle sizes (<63 microns). Very few washoff particles greater than 1000 microns were found. Urban runoff outfall particle size analyses in Bellevue (Pitt and Bissonnette 1984) resulted in a median particle size of about 50 microns. Similar results were obtained in the Milwaukee NURP study (Bannerman et al. 1983).

Particulate residue washoff predictions for Bellevue conditions were made using the Sutherland and McCuen modification of the Yalin equation, and the Sartor and Boyd equation. Three particle size groups (<63, 250-500, and 2000-6350 microns), and three rains, having depths of 5, 10, and 20 mm and 3-hour durations, were considered. The gutter lengths for the Bellevue test areas averaged about 80 m, with gutter slopes of about 4.5 percent. Typical total initial street dirt loadings for the three particle sizes were: 9 g/curb-meter for <63 microns, 18 g/curb-meter for

250-500 microns, and 9 g/curb-meter for 2000-6350 microns. The actual Bellevue net loading removals during the storms was about 45 percent for the smallest particle size group, 17 percent for the middle particle size group, and -6 percent (6 percent loading increase) for the largest particle size group. The predicted removals were 90 to 100 percent using the Sutherland and McCuen method, 61 to 98 percent using the Sartor and Boyd equation, and 8 to 37 percent using the availability factor with the Sartor and Boyd equation. The ranges given reflect the different rain volumes and intensities only. There were no large predicted differences in removal percentages as a function of particle size. The availability factor with the Sartor and Boyd equation resulted in the closest predicted values, but the great differences in washoff as a function of particle size was not predicted.

The Bellevue street dirt washoff observations included effects of additional runoff volume and particulates originating from non-street areas. The additional flows should have produced more gutter particulate washoff, but upland erosion materials may also have settled in the gutters (as noted for the large particles). However, across-the-street dirt loading measurements indicated that much of the street dirt was in the street lanes, not in the gutters, before and after rains. This dirt distribution reduces the importance of these extra flows and particulates from upland areas. The increased loadings of the largest particles after rains were obviously caused by upland erosion, but the magnitude of the settled amounts was quite small compared to the total street dirt loadings.

C. Methodology

Particle dislodgement and transport characteristics at impervious areas were directly measured during the washoff tests. These tests are different from the important Sartor and Boyd (1972) washoff experiments in the following ways:

- They were organized in overlapping factorial experimental designs to identify the most important main factors and interactions.
- Particle sizes were measured down to about one micron (in addition to particulate residue and filterable residue measurements).

- The precipitation intensities were lower in order to better represent actual rain conditions of the upper midwest.
- Observations were made with more resolution at the beginning of the tests.
- Washoff flow rates were frequently measured.
- Emphasis was placed on total street loading, not just total available loading.
- Bacteria population measurements were also periodically obtained (presented in Pitt and McLean 1986).

The washoff tests investigated several important factors and interactions that may affect washoff of different sized particulates from impervious areas:

- street texture
- street dirt loading
- rain intensity
- rain duration
- rain volume

These tests were arranged as an overlapping series of 2^3 factorial tests, one for each particle size and rain total, and were analyzed using standard factorial test procedures described by Box et al. (1978). Non-linear analyses were also used to identify a set of equations to describe the resulting curve shapes. The differences between available and total loads were also related to the experimental factors.

A simple artificial rain simulator was constructed using 12 lengths of "soaker" hose, suspended on a wooden framework about one meter above the road surface. "Rain" was applied by connecting the hoses to a manifold, having individual pressure valves to balance the rain for different areas, which was in turn connected to a fire hydrant. The flow rate needed for each test was calculated based on the desired rain intensity and the area covered. The flow rates were carefully monitored by using a series of ball flow gauges before the manifold. The distributions of the test rains over the study areas were also monitored by placing about 20 small graduated cylinders over the area during the rains. In order to keep the drop sizes representative of sizes found during natural rains, the surface tension of the water drops hanging on the plastic soaker hoses was reduced by applying a light coating of teflon spray to the hoses.

It was difficult to obtain even distributions of rain during the light rain tests using the manifold, so

a single hose was used that was manually walked back and forth over the test area during the smaller rain tests (three people took 30-minute shifts). To keep evaporation reasonable for the rain conditions, the test sites were also shaded during sunny days. Blank water samples were also obtained from the manifold for background residue analyses. The filterable residue of the "rain" water (about 185 mg/L) could cause substantial errors when predicting washoff.

The areas studied were about 3 by 7 meters each. The street side edges of the test areas were edged with plywood, about 30 cm in height and imbedded in thick caulking, to direct the runoff towards the curbs with minimal leakage. All runoff was pumped continuously from downstream sumps (made of caulking and plastic sand bags) to graduated 1000 L Nalgene containers. The washoff samples were obtained from the pumped water going to the containers every 5 to 10 minutes at the beginning of the tests, and every 30 minutes near the tests' conclusions. Final complete rinses of the test areas were also conducted (and sampled) at the tests' conclusions to determine total loadings of the monitored constituents.

The samples were analyzed for total residue, filtrate residue, and particulate residue. Runoff samples were also filtered through 0.4 micron filters and microscopically analyzed (using low power polarized light microscopes to differentiate between inorganic and organic debris) to determine particulate residue size distributions from about 1 to 500 microns. The runoff flow quantities were also carefully monitored to determine the magnitude of initial and total rain water losses on impervious surfaces, as reported earlier in Section 5.

D. Controlled Street Dirt Washoff Test Results

Residue Concentrations and Particle Sizes in Runoff

A series of eight controlled washoff tests were conducted in Toronto as part of this research to identify the most important factors that affect particulate washoff and to develop washoff algorithms for use in urban runoff models. This subsection presents the observed runoff residue concentration and particle size information.

The two extreme levels of the cleanliness factor were for dirty streets (the first sampling run at a site, with obviously dirty streets) and clean streets (the same site, two days after completely flushing the street surface). The two extreme values for the texture category were for smooth and rough textured streets. The extreme values of rain intensity were controlled by the application of artificial rain. The eight tests represented all possible combinations of these three factors (except for low rains on dirty, smooth streets, which were actually cleaner than anticipated). The following list summarizes the experimental conditions used, along with the test codings:

- o Rain intensity:
  - L: light rain intensity, 2.9 to 3.2 mm/hr.
  - H: high rain intensity, 11.0 to 12.2 mm/hr.

- o Street cleanliness particulate residue loadings (half street widths were 3.2 to 3.7 m):
  - C: clean streets, 1.7 to 2.6 g/m^2
  - D: dirty streets, 10.5 to 12.6 g/m^2

- o Pavement texture (see Section 5):
  - R: rough streets, 1.1 mm detention storage

S: smooth streets, 0.3 to 0.4 mm detention storage

As an example, the LCR test had a rain intensity of 2.9 mm/hr (light), a half street width of 3.5 m, and a total particulate residue loading value of 2.25 g/m^2 (clean), and a street texture expressed as 1.1 mm detention storage (rough).

Figures 9.3 through 9.5 are plots of observed residue concentrations against rain depths, for the eight washoff tests. The total residue concentrations varied from about 25 to 3000 mg/L, with an obvious decrease in concentrations with increasing rain depths. No concentrations greater than 500 mg/L occurred after about 2 mm of rain, while all concentrations after about 10 mm of rain were less than 100 mg/L. Generally, total residue concentrations appear to be independent of the test conditions. A similarly wide range in runoff concentrations was observed for particulate residue (>0.4 microns) (Figure 9.4), from about 1 to 3000 mg/L. Again, a downward trend of concentrations is seen with increasing rain depths, but the data scatter is larger because of some small concentrations. After

Figure 9.3 Total Residue Concentrations During Street Dirt Washoff Tests

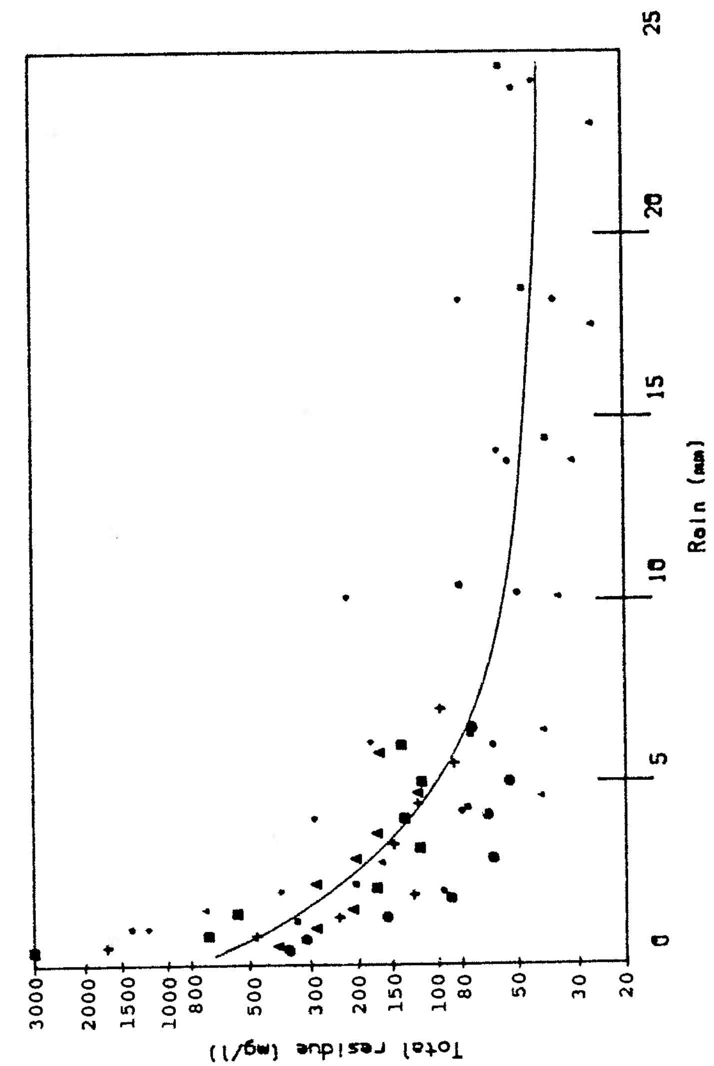


Figure 9.4 Particulate Residue Concentrations During Street Dirt Washoff Tests

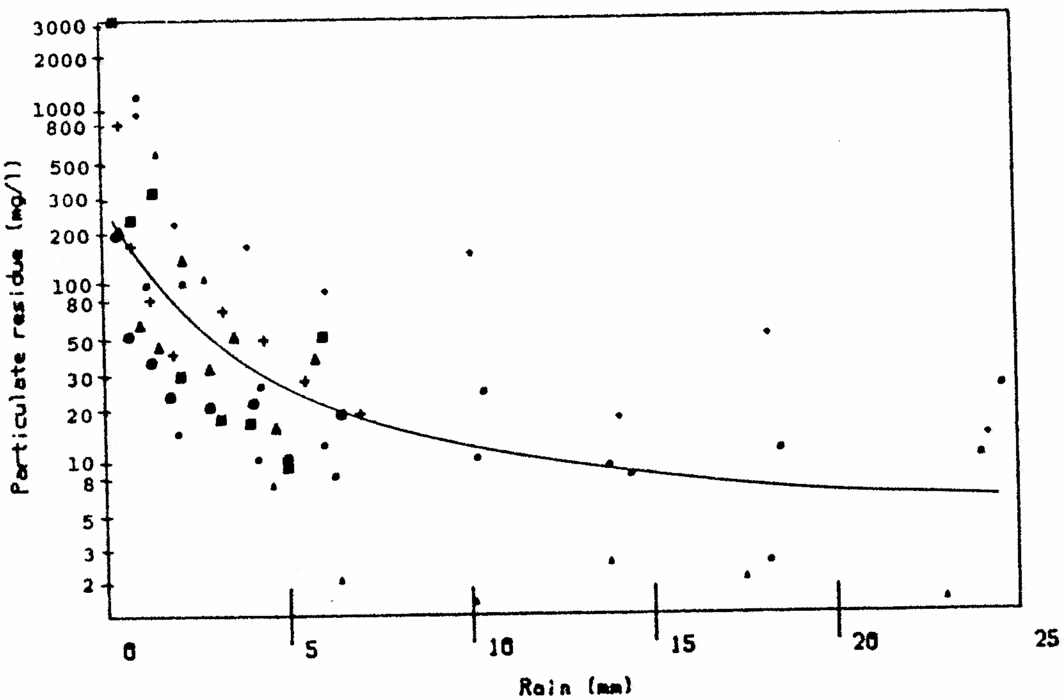


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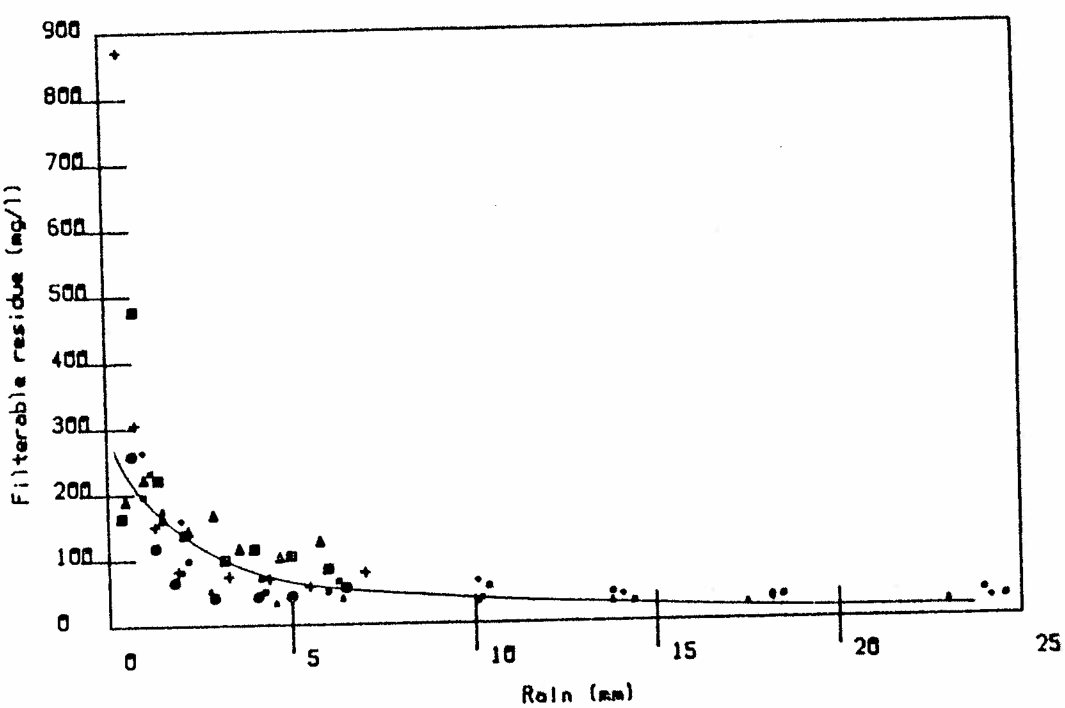
Figure 9.5 Filterable Residue Concentrations During Street Dirt Washoff Tests



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10 mm, the maximum particulate residue concentrations observed were about about 200 mg/L.

The filterable residue (<0.4 microns) (Figure 9.5) concentrations (after subtracting background concentrations) ranged from about 20 to 900 mg/L, comprising a surprisingly large percentage of the total residue loadings. Figure 9.6 is a plot of the total residue portion that was filterable for the different tests. For small elapsed rain depths, filterable residue comprised from less than 5 to about 90 percent of the total residue. For larger rains, the range was less, with filterable residue making up 70 to 90 percent of the total residue, except for the high rain intensity test on dirty smooth streets (HDS test), where the filterable residue was only about 30 to 50 percent of the total. After 10 mm of elapsed rain depth, the filterable residue concentrations were all less than about 50 mg/L. The domestic water used during these tests had filterable residue concentrations of about 185 mg/L which would have substantially added to the washoff discharges if not subtracted.

Particle size analysis was also conducted on the particulate residue washoff samples. Figures 9.7 through 9.9 are examples of particle size distributions

Figure 9.6 Filterable Residue as a Percentage of Total Residue During Street Dirt Washoff Tests

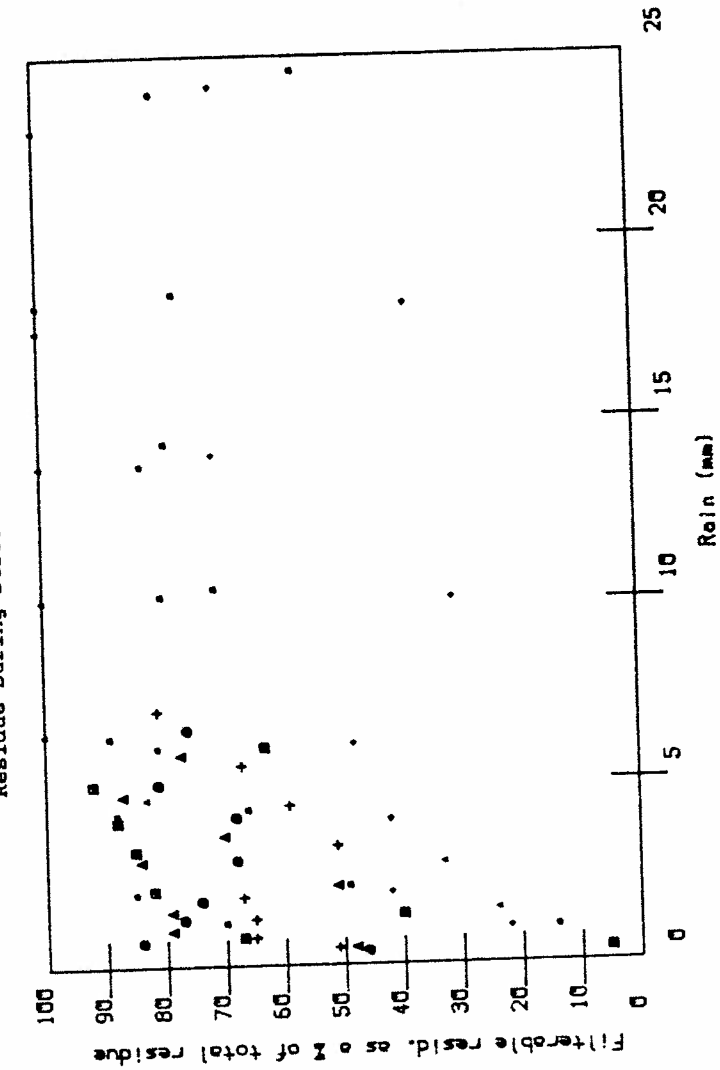


Figure 9.7 Particle Size Distribution (by volume) for HCS Test

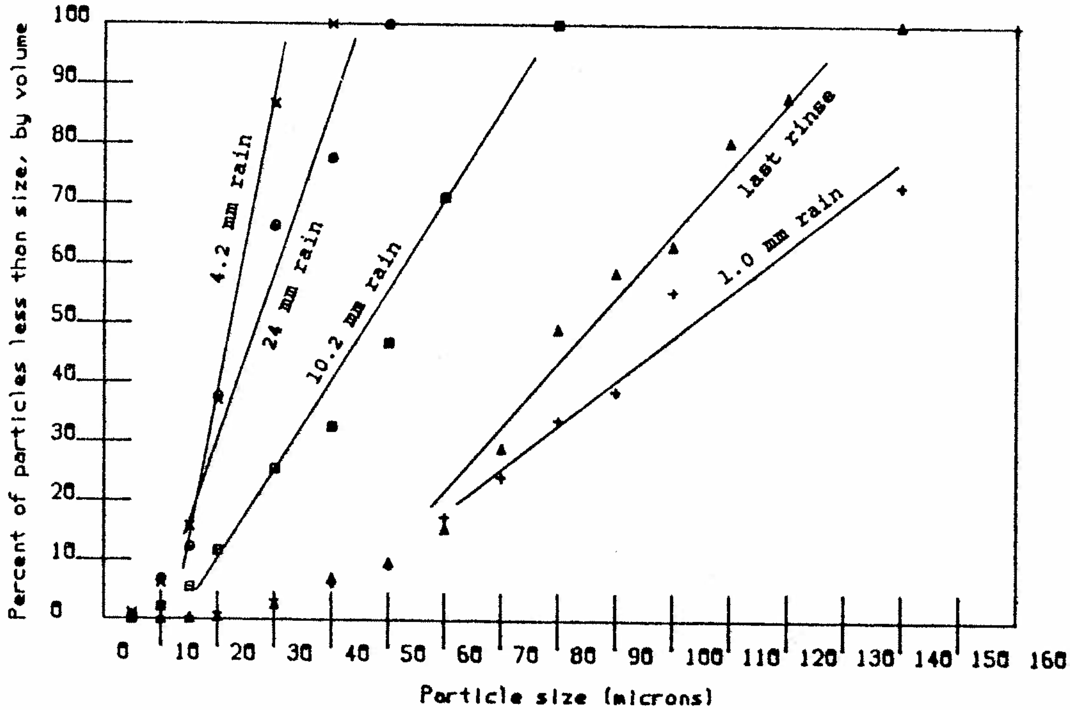


Figure 9.8 Particle Size Distribution (by volume) for HDS Test

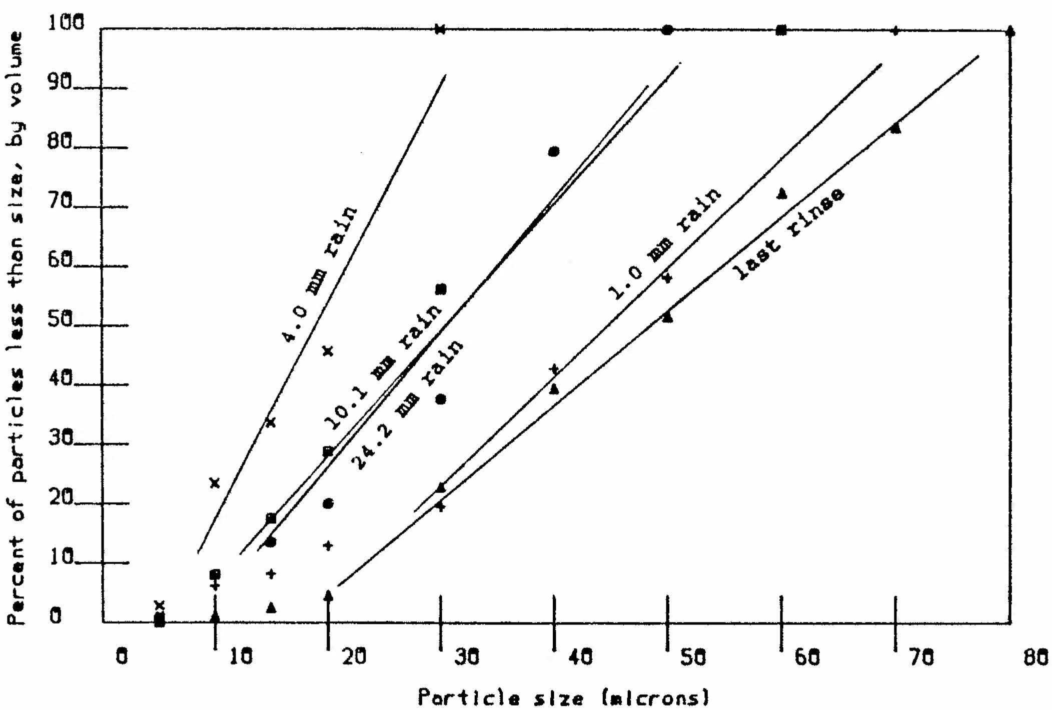
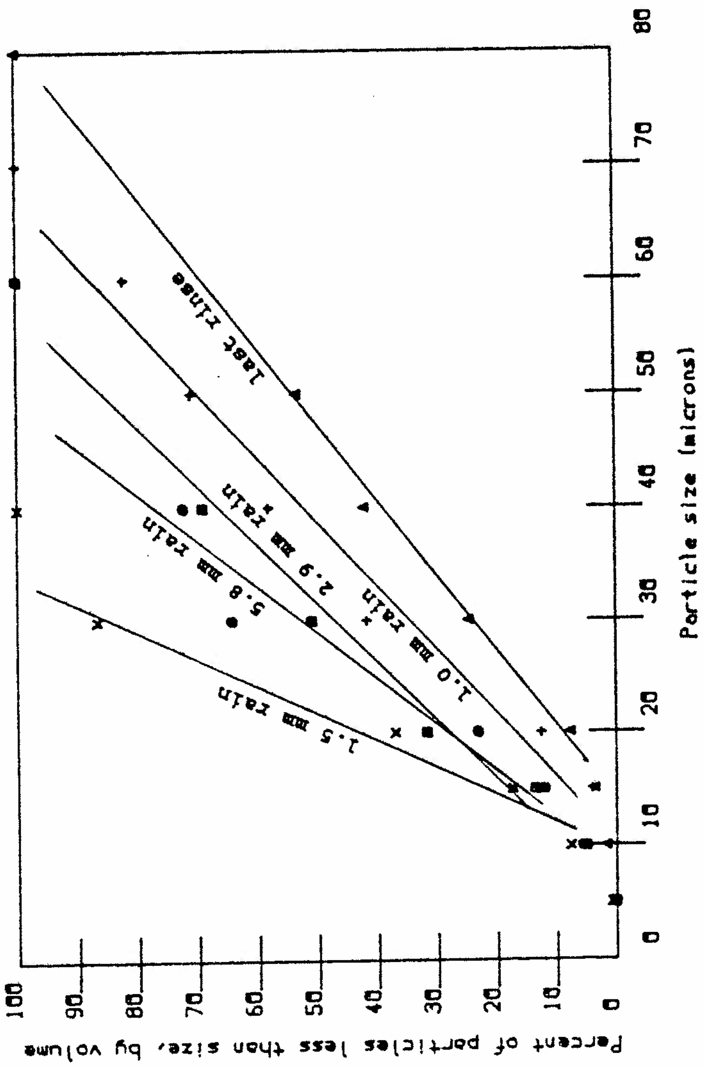


Figure 9.9 Particle Size Distribution (by volume) for LCR Test



for three tests. The plots show the percentage of the particles that are less than various sizes, by measured particle volume (similar to weight). The plots also indicate median particle sizes of about 20 to 100 microns, depending on when the sample was obtained during the washoff tests. All three distributions show surprisingly similar trends of particle sizes with elapsed rain depth. The median size for the sample obtained at about 1 mm of rain was much greater than for the samples taken after more rain. The median particle sizes of material remaining on the streets after the washoff tests were also much larger than for most of the runoff samples, but were quite close to the initial samples' median particle sizes. The washoff water at the very beginning of the test rains therefore contained many more larger particles than during later portions of the rains. Also, a substantial amount of larger particles remained on the streets after the test rains. Most street runoff waters during test rains in the 5 to 15 mm depth category had median particulate residue particle sizes of about 10 to 50 microns. However, filterable residue (less than 0.4 microns) made up most of the total residue washoff for elapsed rain depths greater than about 5 mm.

These particle size distributions indicate a much greater importance of smaller particles than previous tests. As an example, the Sartor and Boyd (1972) washoff tests found median particle sizes of about 150 microns which were typically three to five times larger than were found during these tests. They also did not find any significant particle size distribution differences for different rain depths (or rain durations or rain intensities), in contrast to these tests. These earlier tests did not measure filterable residue, so the actual particle size distributions would actually be shifted to smaller particle sizes.

<u>Washoff Equations for Individual Tests</u>

The factorial experiments allowed many values to be obtained. Washoff values for total residue, filterable residue (particles less than 0.4 microns in diameter), and particulate residue (particles greater than 0.4 microns in diameter) were obtained for nine time periods. The time periods were at approximately 5, 10, 20, 30, 50, 70, 90, and 120 minutes, plus the final complete rinse.

The particulate washoff values were expressed in units of grams per square meter and grams per curb-

meter, concentrations (mg/L), and the percent of the total initial loading washed off. This subsection presents the basic washoff plots and the individually fitted washoff equations for these tests for total, particulate, and filterable residue. A later subsection describes the factorial analyses that were used to identify significant test factors affecting these washoff equation parameters and the development of procedures to help select the equation parameters.

Plots of accumulative washoff (similar in form to the Sartor and Boyd washoff plots shown on Figure 9.2) are shown on Figures 9.10 through 9.17. These plots show the asymptotic washoff values observed in the tests, along with the measured total street dirt loadings. The maximum asymptotic values are the "available" street dirt loadings. The measured total loadings are seen to be several times larger than these "available" loading values. As an example, the asymptotic available total residue value for the HDS test (Figure 9.16) was about 3 g/m^2, while the total load on the street for this test was about 14 g/m^2, or about five times the available load. The differences between available and total loadings for the other tests were even greater, with the total loads typically

Figure 9.10 Accumulative Washoff Plots for HCR Test

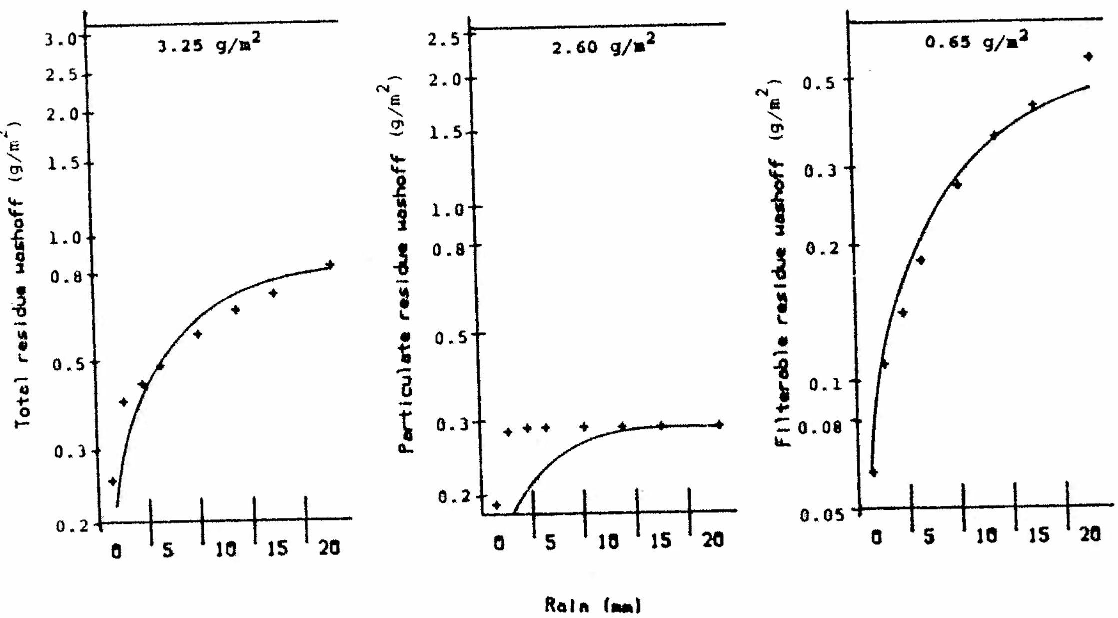


Figure 9.11 Accumulative Washoff Plots for LCR Test

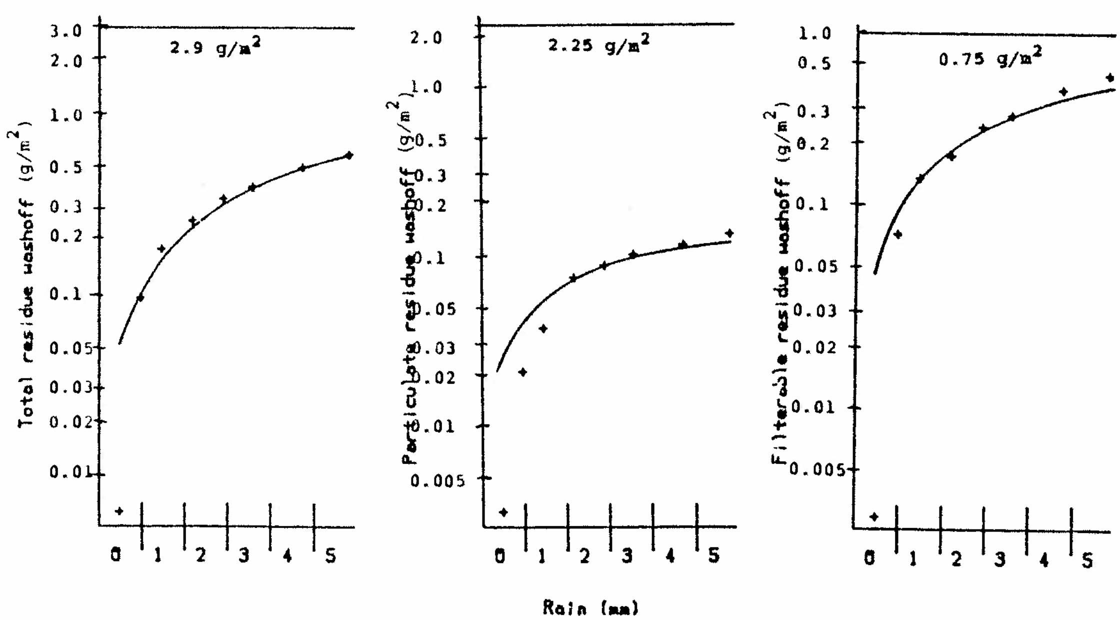


Figure 9.12 Accumulative Washoff Plots for HDR Test

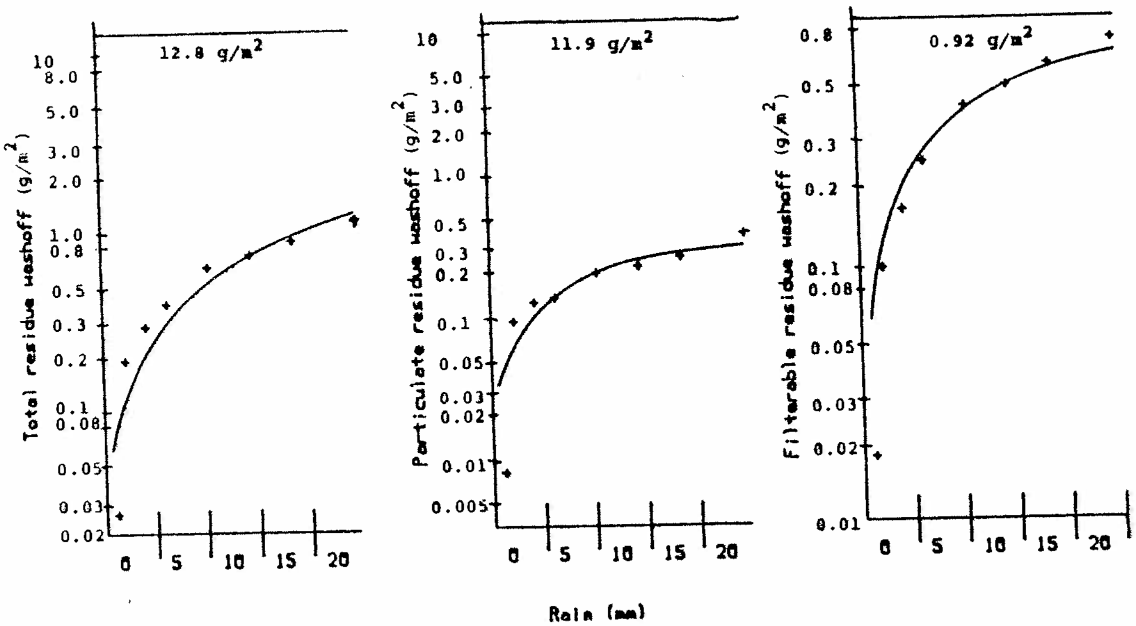


Figure 9.13 Accumulative Washoff Plots for LDR Test

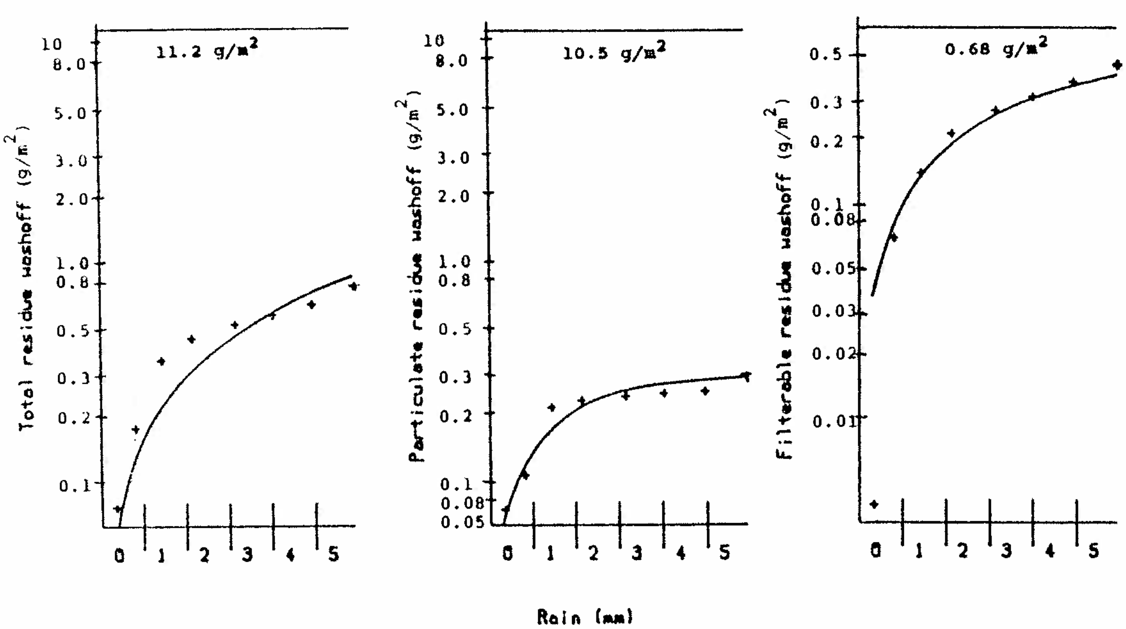


Figure 9.14 Accumulative Washoff Plots for HCS Test

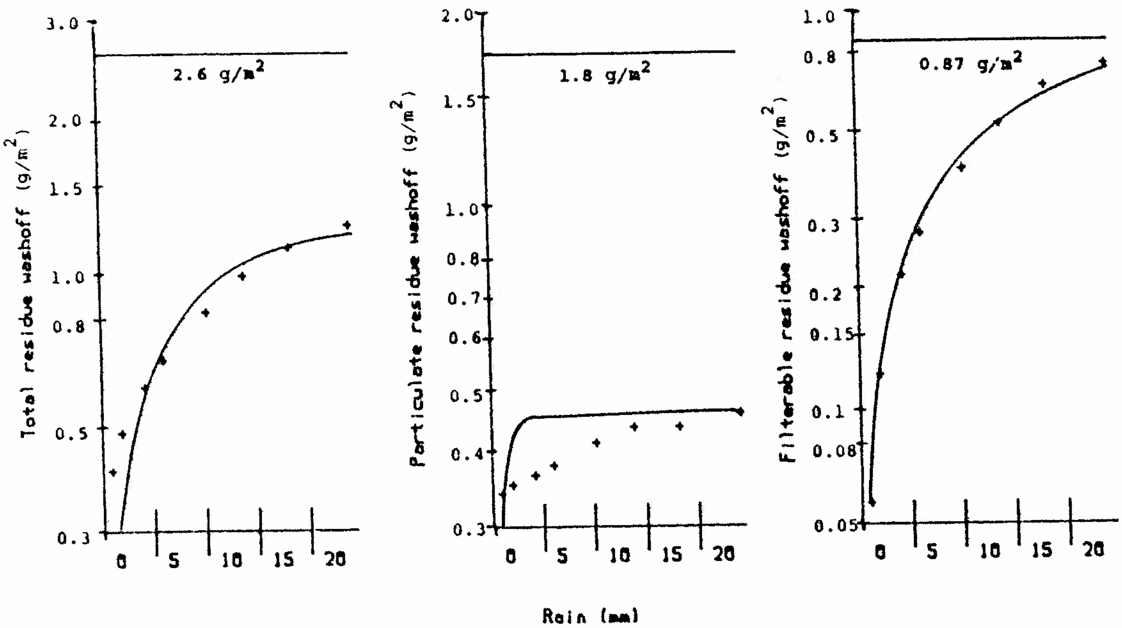


Figure 9.15 Accumulative Washoff Plots for LCS Test

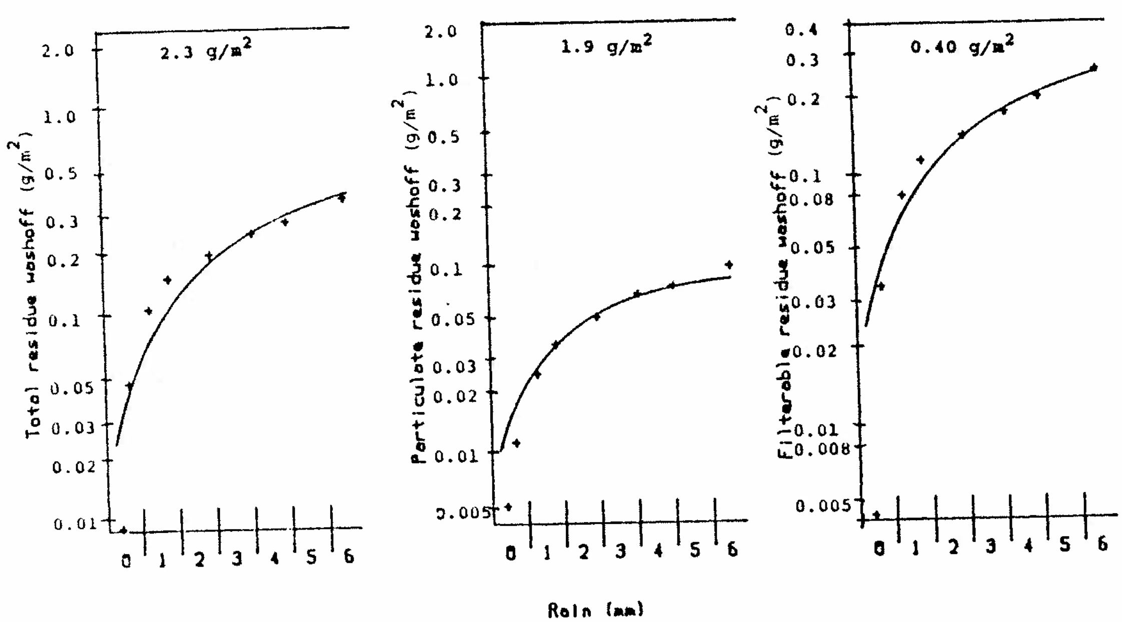


Figure 9.16 Accumulative Washoff Plots for HDS Test

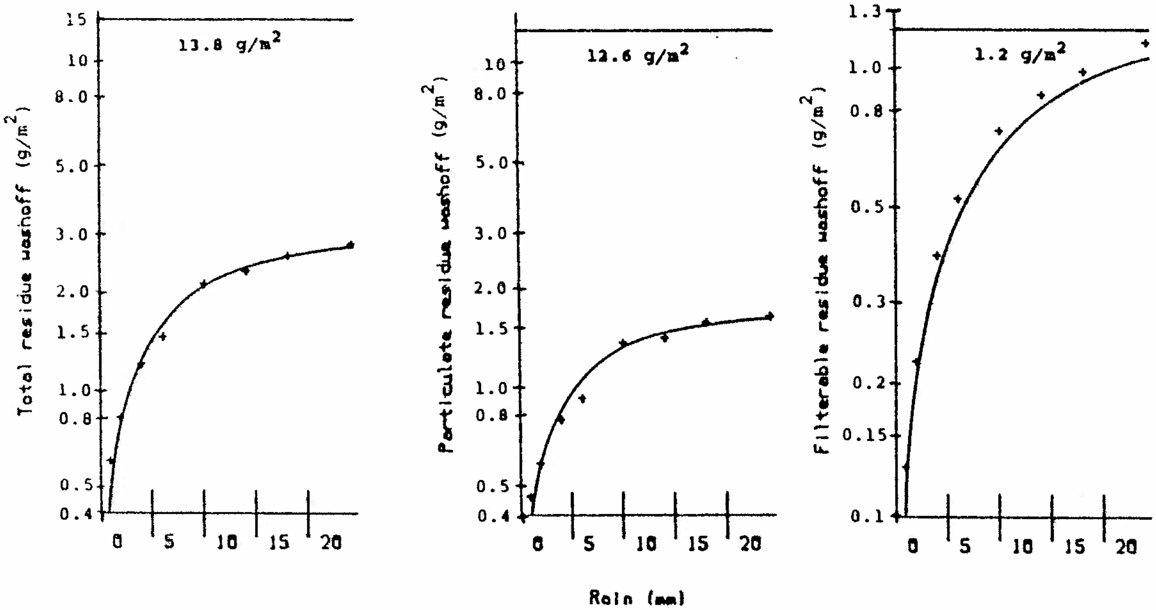
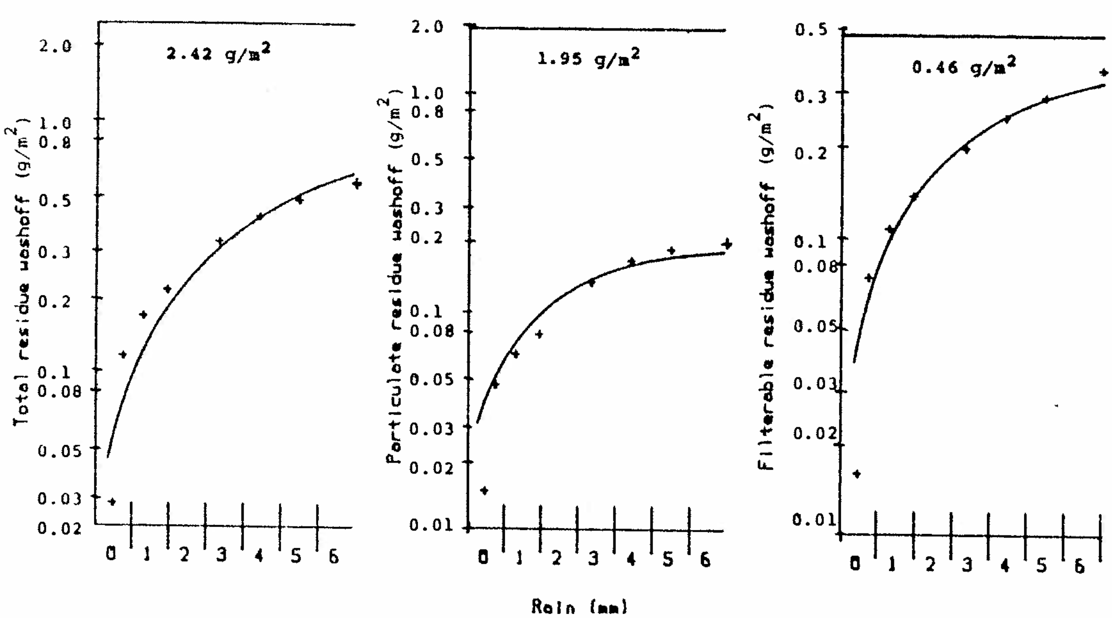


Figure 9.17 Accumulative Washoff Plots for L(D)CS Test



about ten times greater than the available loads. The total loading and available loading values for filterable residue were quite close, indicating almost complete washoff of the very small particles. However, the differences between the two loading values for particulate residue were much greater.

The actual data are shown in these figures along with the selected nonlinear models using the Sartor and Boyd exponential washoff equation described earlier. In many cases (LCR, HDR, HCS, LCS, and LDCS), the fitted washoff equations greatly over-predicted particulate residue washoff during the very small rains (usually less than 1 to 3 mm in depth). In all cases, the fitted washoff equations described particulate washoff very well for rains greater than about 10 mm in depth. In critical water quality modeling applications, where the small storms can be significant pollutant contributors, correction factors may be needed to better describe washoff during these small rains that do not behave according to the exponential washoff model.

Tables 9.3 through 9.5 present the equation parameters for each of the eight washoff tests for total, particulate, and filterable residue. The model proportionality terms (k) were selected using the

Table 9.3 Total Residue Washoff for Individual Tests

	N_o = measured total residue loading			N_a = "available" total residue loading		
Test	N_o	Calculated k	Standard Error For k	N_a	Calculated k	Standard Error For k
HCR	3.25	0.016	0.002	0.84*	0.145*	0.018
LCR	2.99*	0.038*	0.001	0.58	0.304	0.032
HDR	12.82*	0.004*	<0.001	1.14	0.078	0.006
LDR	11.22*	0.013*	0.001	0.74	0.383	0.024
HCS	2.62	0.033	0.005	1.21*	0.146*	0.021
LCS	2.32*	0.026*	0.001	0.35	0.301	0.024
HDS	13.82	0.012	0.001	2.74*	0.138*	0.008
L(D)CS	2.42*	0.042*	0.002	0.57	0.300	0.024

*These model parameters resulted in the best fitting relationships, based on residuals analyses.

Table 9.4 **Particulate Residue Washoff for Individual Tests**

(N_o = "available" particulate residual loading)

Test	N_o	Calculated k	Standard Error for k	Ratio of "Available Load to Total Load
HCR	0.295	0.832	0.064	0.11
LCR	0.138	0.344	0.038	0.061
HDR	0.375	0.077	0.008	0.032
LDR	0.291	0.619	0.052	0.028
HCS	0.462	1.007	0.321	0.26
LCS	0.091	0.302	0.024	0.047
HDS	1.66	0.167	0.015	0.13
L(D)CS	0.209	0.335	0.031	0.11

Table 9.5 **Filterable Residue Washoff for Individual Tests**

(N_o = measured total initial filterable residue loading)

Test	N_o	Calculated k	Standard Error for k
HCR	0.651	0.061	0.004
LCR	0.745	0.139	0.006
HDR	0.915	0.058	0.002
LDR	0.680	0.163	0.006
HCS	0.871	0.070	0.003
LCS	0.395	0.154	0.007
HDS	1.223	0.085	0.002
L(D)CS	0.463	0.183	0.008

NONLIN module of the SYSTAT (Version 3, May 1986) computer package (SYSTAT Inc., Evanston, Ill.). The preferred models were selected based on analyses of residual plots for many alternative models. The standard errors for the fitted k values are also shown on these tables. The standard errors were found to be 5 to 40 times less than the selected k values. The N_o terms were directly measured during the tests.

The N_o values selected for the total loading forms of the washoff models were the total measured street loadings of the form of residue being considered. For the available loading forms of the models, the N_o values were the last measured loading values obtained during the washoff tests. These available N_o values were confirmed using the NONLIN curve fitting module to predict smoothed asymptotic N_o available loading values simultaneously with the k terms. In all cases, the last measured loading values were within 10 percent of the calculated asymptotic available loading values. The measured values for the N_o terms (instead of the calculated values) were preferred as they best represented the procedures that would be typically used in calibrating an urban runoff model, without conducting washoff tests.

If a selected model requires available loading values instead of the total loading values, then a procedure must be used to adjust the total loading values (such as attempted by the availability term in STORM and SWMM). In all cases, the k term must be appropriate for the model form. As shown in the following paragraphs, the use of an available loading value for N_o requires the use of a substantially larger k term compared to using the total loading value.

The total residue models were fitted using both total and available residue values to show the differences in the proportionality terms (k) for each loading type. In three cases (HCR, HCS, and HDS), the available residue form of the equations provided much better model residual analyses and were therefore preferred over the candidate equations using total loadings. The k values varied greatly (by about 5 to 30 times), depending on the use of total or available loadings.

Some of the attempts at fitting outfall data to the washoff model used total street dirt loading values, while the Sartor and Boyd values were based on available loadings. Obviously, this difference in loading definition easily could have been responsible for causing such different k values to be identified. The available loading forms of the equations for these washoff tests produced the largest k values (0.078 to 0.38), and are similar to the reported Sartor and Boyd value of 0.18 that is used as a "default" in many urban runoff models. The total loading model k terms are much smaller (0.004 to 0.042) and are close to those reported by Novotny (undated) (0.019 to 0.026) using Milwaukee NURP street dirt washoff observations and actual measured total street dirt loadings.

Selecting the appropriate k term for the correct form of N_o is critical. As an example, the rain volume needed to produce 90 percent washoff can be calculated using the standard washoff equation as follows:

$$N = N_o e^{-kR}$$

for 90 percent washoff, $N = 0.1 N_o$, and

$$0.1 N_o = N_o e^{-kR}, \text{ or}$$

$$0.1 = e^{-kR}, \text{ and}$$

$$(1/k) \log_e (0.1) = R, \text{ therefore}$$

$$R = 2.303/k \text{ for 90 percent washoff.}$$

For a k value of 0.3 (the LCS model for available total residue loadings shown on Table 9.3), the rain needed for 90 percent washoff would be 8 mm. This rain would produce a washoff total of about 0.32 g/m^2 using the appropriate available N_o loading of 0.35 g/m^2. If the k value of 0.026 was used instead (appropriate for the total loading form of the LCS model), a rain of almost 90 mm would be needed for 90 percent washoff (more than ten times the rain depth predicted using the larger k value). In this case however, a total N_o value of 2.32 g/m^2 should be used, producing a washoff quantity of about 2.1 g/m^2 (more than 6.5 times the total residue washoff produced above). In all cases, the fitted models should obviously be used with caution beyond the test conditions. The 8 mm rain prediction is well within the test conditions, while the 90 mm rain prediction is almost four times the maximum rain used in these washoff tests. Other relationships between k values and rain quantities (mm) to produce specific percent washoffs are as follows:

Percent washoff	Rain needed (mm)
99.9	6.908/k
99	4.605/k
95	2.996/k
90	2.303/k
75	1.386/k
50	0.693/k
25	0.288/k
10	0.105/k

From these relationships, it is obvious that washoff occurs faster for larger k values (the washoff curves presented in Figures 9.10 through 9.17 would be steeper for larger k values if the figures were plotted without log scales).

The selected particulate residual washoff models were all based on the available loading model form because of superior model residual behavior. Therefore, an additional relationship is needed to predict available loading from total observed loading. The available particulate residue loadings ranged from about 3 to 25 percent (with an average of about 10 percent) of the total particulate residual loadings. A following subsection presents factorial models to better explain this relationship for significant test conditions.

The filterable residue washoff models, however, were all based on total measured filterable residue loadings. These different preferred model forms for particulate and filterable residue were most likely caused by the differences in washoff efficiencies for different sized particles. Particulate residues were not nearly as efficiently removed during the washoff tests and were better related to much reduced "available" particulate residue loading values. Filterable residues in contrast, were much more efficiently removed and related well to total loadings (not much filterable residue was left on the streets after the washoff tests, making the available loadings very similar to the total loadings for filterable residue).

Street Dirt Accumulation Rates

Washoff of street dirt is very dependent on the available street dirt loading. Therefore, a series of street dirt accumulation measurements were conducted as part of this research to supplement these washoff experiments. The street dirt data are contained in the report prepared for the Ontario Ministry of the Environment (Pitt and McLean 1986).

An industrial street with heavy traffic (Norseman) and a residential street with light traffic (Glen Roy) were monitored about twice a week for three months. At the beginning of this period, intensive street cleaning (one pass per day for each of three consecutive days) was conducted to obtain reasonably clean streets. Street dirt loadings were then monitored every few days to measure the accumulation rates of street dirt.

The Toronto accumulation rate data compare favorably with that of other sites for similar conditions. The most important factors affecting the street dirt accumulation rate was found to be land use, while the initial loading and maximum loading values possible were most influenced by street texture and condition.

The particulate accumulation rate for the residential street was found to be much less than the accumulation rate for the industrial street, probably because of the decreased traffic and other land use activities that contributed to dirty streets (such as tracking dirt from unpaved driveways and parking areas onto the streets). When data from many locations are

studied, it is apparent that smooth streets have substantially less loadings at any accumulation period than rough streets for the same land use.

Street dirt particulate loadings were quite high before the initial intensive street cleaning period during these measurements and were reduced to their lowest observed levels immediately after the last street cleaning. After street cleaning, the loadings on the industrial street increased much faster than for the residential street. Right after intensive cleaning, the street dirt particle sizes were also similar for the two land uses. However, the loadings of larger particles on the industrial street increased at a much faster rate than on the residential street, indicating more erosion or tracking materials being deposited on the industrial street.

First degree (for the residential street) and second degree (for the industrial street) polynomial linear regression equations were derived (using SYSTAT) from the observed accumulation data and were integrated into the Source Loading and Management Model (SLAMM) to describe the accumulation of particulates on street surfaces. These equations were related to street dirt accumulation processes by Pitt (1979). The following subsection describes the washoff relationships that were developed for these particulates.

E. Development of Particulate Washoff Models for Significant Factors

As described earlier, the washoff tests were designed as a complete 2^3 factorial experiment to investigate the effects of rain intensity, street cleanliness, and street texture on particulate washoff. Eight tests were conducted to allow the necessary factorial calculations. However, after the monitoring data were collected, it was found that one of the intended "dirty" street tests (coded as LDS; light rain intensity, dirty street, smooth texture), actually had initial dirt loadings much less than expected and were actually very similar to the dirt loadings for the "clean" tests. This was probably due to the smooth street being unable to retain high street dirt loadings due to its smooth texture. Therefore a simplified version of the factorial calculations was used that recognized the observations from this experiment as duplicates of the LCS experimental observations.

For this simplified factorial analysis, each calculation required three separate analyses to examine all possible main factor and two-way factor interactions, as shown on Table 5.3. The three-way factor interaction was not investigated in these calculations, but the outcomes of the main factor models could be confirmed. In these alternative analyses, pooled standard error values were calculated and used to identify the significant model factors. Only the factor effects greater than the standard error values were assumed to be significant.

Tables 9.6 through 9.8 summarize the significant factorial models describing the initial residue loadings (N_o) and the proportionality constant (k) for total residue, particulate residue, and filterable residue, using the individual parameter values shown in Tables 9.3 through 9.5. The parameter values for total residue for the measured total residue models were used for these tests to reduce confusing the factorial analysis by comparing different sets of parameters. Table 9.7 for particulate residue also shows a factorial model relating available loading to total loading. The significant factors affecting these equation parameters varied for each form of residue

Table 9.6 Summarized Results for Factorial Tests Investigating Total Residual Washoff

1. N_o measured total residual loading before washoff (g/m^2)

   $C = 10.14 \pm 0.44$*

   $\hat{Y} = 7.85 + 5.07\ (C)$

   C- (clean streets): $\hat{Y} = 2.78\ g/m^2$
   C+ (dirty streets): $\hat{Y} = 12.92\ g/m^2$

   * Possible weak two-way interactions of CI and CT may occur instead of C alone.

2. k, proportionality constant (for rain in mm)

   $C = -0.020 \pm 0.007$
   $I = -0.014 \pm 0.009$

   $\hat{Y} = 0.020 - 0.010\ (C) - 0.007\ (I)$

   C+I+ (dirty, high inten.): $\hat{Y} = 0.003$
   C+I- (dirty, low inten.): $\hat{Y} = 0.017$
   C-I+ (clean, high inten.): $\hat{Y} = 0.023$
   C-I- (clean, low inten.): $\hat{Y} = 0.037$

Table 9.7 Summarized Results for Factorial Tests Investigating Particulate Residual Washoff

1. N_o, "available" particulate residue loading before washoff (g/m^2)

   $I = 0.53 \pm 0.32$*

   $\hat{Y} = 0.45 + 0.27(I)$

   I+ (high intensity): $\hat{Y} = 0.72\ g/m^2$
   I- (low intensity) : $\hat{Y} = 0.18\ g/m^2$

   * Possible weak two-way interaction of IT plus a two-way interaction of CT may occur instead of I.

2. Ratio of "available" particulate residue loadings to total particulate residue loadings.

   $I = 0.08 \pm 0.04$
   $T = -0.08 \pm 0.05$

   $\hat{Y} = 0.097 + 0.04(I) - 0.04(T)$

   I+T+ (high and rough) : $\hat{Y} = 0.10$
   I+T- (high and smooth): $\hat{Y} = 0.18$
   I-T+ (low and rough) : $\hat{Y} = 0.02$
   I-T- (low and smooth) : $\hat{Y} = 0.10$

3. k, proportionality constant (for rains in mm)

   $IC = -0.55 \pm 0.054$

   $\hat{Y} = 0.46 - 0.28(IC)$

   I+C- (high and clean) or I-C+ (low and dirty): $\hat{Y} = 0.74$
   I+C+ (high and dirty) or I-C- (low and clean): $\hat{Y} = 0.18$

Table 9.8 Summarized Results for Factorial Tests Investigating Filterable Residual Washoff

1. N_o, measured total initial filterable residue loading before washoff

$$I = 0.33 \pm 0.13^{*}$$
$$C = 0.30 \pm 0.14^{**}$$

$$\hat{Y} = 0.74 + 0.17(I) - 0.15(C)$$

I+C+ (high inten./dirty):	$\hat{Y} = 1.06$
I+C- (high inten./clean):	$\hat{Y} = 0.76$
I-C+ (low inten./dirty):	$\hat{Y} = 0.72$
I-C- (low inten./clean):	$\hat{Y} = 0.42$

* Possible weak two-way interaction of IT may occur instead of I alone.

** Possible weak two-way interaction of CT may occur instead of C alone.

2. k, proportionality constant (for rain in mm)

$$I = -0.089 \pm 0.019$$
$$T = -0.018 \pm 0.010$$

$$\hat{Y} = 0.114 - 0.045(I) - 0.009(T)$$

I+T+ (high inten./rough):	$\hat{Y} = 0.060$
I+T- (high inten./smooth):	$\hat{Y} = 0.078$
I-T+ (low inten./rough):	$\hat{Y} = 0.15$
I-T- (low inten./smooth):	$\hat{Y} = 0.16$

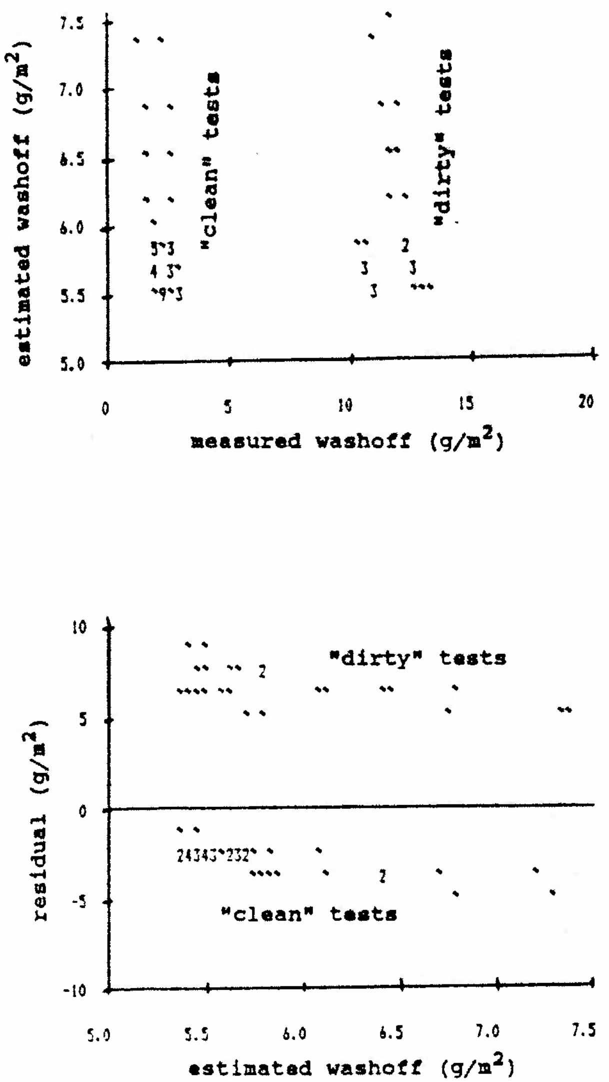
investigated, further confirming the important differences in washoff for particles of differing sizes.

The total residue initial loading parameter (N_o) was most affected by street cleanliness, as expected. Figure 9.18 is a residual plot of a NONLIN model analysis using all test site data combined to predict N_o. The two major families of residuals are associated with the "clean" tests having negative residuals, and the "dirty" tests having positive residuals.

The factorial relationship between available and total particulate residue loadings shown on Table 9.7 indicate significant rain intensity and street texture effects. As rain intensity increased, the percentage of total loading available for ultimate washoff also increased. Conversely, for the rough street texture tests, the percentage available decreased.

Both washoff model parameters for filterable residue were affected by rain intensity; the initial loading parameter was affected by street cleanliness and the proportionality constant by street texture. Therefore, it was not practical to combine any of the filterable residue washoff test results to produce simpler models. The observed residuals using the

Figure 9.18 Residual Plots for Total Residue Washoff Model (all tests combined)



individual washoff models for filterable residue were all very small, ranging from about -0.06 to 0.05 g/m^2, compared to washoff values for 10 mm rains of 0.3 to 1 g/m^2.

Selected washoff test results were combined for total residue and particulate residue, based on the factorial analyses, for new NONLIN model analysis. Tables 9.9 and 9.10 summarize these combined results. Combining total residue test results based on cleanliness and rain intensity produced the best residual analyses. The N_o values for the two dirty groups were quite similar, as were the two values for the clean groups. The k terms, however, did vary based on intensity. Lower rain intensities increased the k values by about 50 percent. This would have the effect of increasing the washoff rates (if the N_o value remained constant), which seems unreasonable. However, the N_o values for the low intensity tests were smaller for each cleanliness subcategory, resulting in less actual washoff for comparable rain depths. Similarly, the clean group had substantially greater k values and much smaller N_o values, also resulting in less washoff from clean streets as compared to dirty streets for similar rain depths.

Table 9.9 Total Residue Washoff for Combined Tests

Test Category	N_o = measured total residue loading: N_o	Calculated k	Standard Error For k	N_o = "available" residual loading: N_o	Calculated k	Standard Error For k
1) Cleanliness and rain intensity						
C+I+ (dirty, high)	13.3	0.008	0.001	1.94	0.12	0.017
C+I- (dirty, low	11.2	0.012	<0.001	0.74	0.38	0.024
C+I+ (clean, high)	2.94	0.023	0.004	1.03	0.15	0.017
C-I- (clean, low)	2.58	0.036	0.006	0.50	0.30	0.033
2) Cleanliness only						
C+ (dirty)	12.6	0.006	0.002	1.54	0.17	0.045
C- (clean)	2.72	0.022	0.004	0.71	0.27	0.052
3) All tests combined	5.35	-0.013	0.013	-	-	-

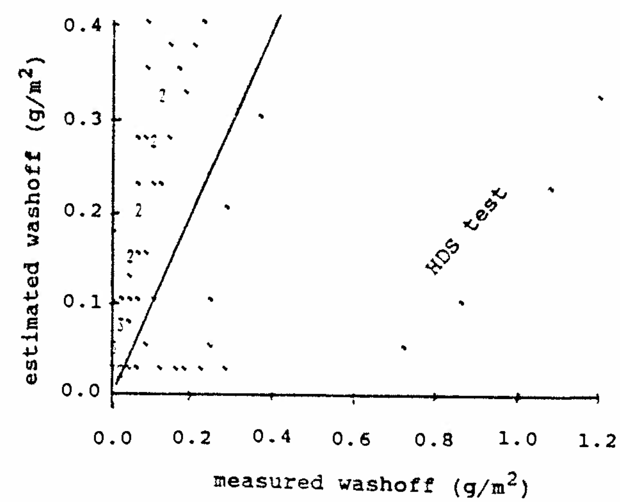
Table 9.10 Particulate Residue Washoff for Combined Tests
(N_o = "available" particulate residual loading)

Test	N_o	Calculated k	Standard Error For k
1) rain intensity and texture			
I+I+ (high rough)	0.335	0.195	0.074
I+I+ (high smooth)	1.061	0.206	0.046
I-I+ (low rough)	0.215	0.492	0.046
I-I- (low, smooth)	0.150	0.328	0.067
2) Intensity only			
I+ (high)	0.70	0.206	0.053
I- (low)	0.18	0.405	0.041
3) All tests combined	0.44	0.377	0.090
4) All test, except HDS	0.27	0.463	0.085
5) All tests, except HDS and HDR	0.25	0.613	0.060

The combined test results for particulate residue washoff is shown on Table 9.10. High rain intensities had greater available residue loadings, and smaller k values, compared to low rain intensity tests, as predicted from the factorial calculations. However, the combined test residual analyses indicated some washoff behavior that was not readily apparent from the factorial tests. Figure 9.19 contains plots of the combined test NONLIN model for particulate residue and show the unusual behavior of the HDS test, as compared to the other tests. The HDS residuals indicate much greater washoff than the other tests, as expected. The combination of high rain intensities, dirty streets, and smooth textures would seem to cause the most washoff of all the test conditions, as observed.

Figure 9.20 contains residual plots for the washoff model using all of the tests combined, except for HDS. Again, unusual behavior is indicated in one of the tests: HDR. The residuals for this test are only about 1/3 as large as the previous HDS residuals, but still indicate the possibility of significantly increased washoff. Both of these tests had high intensity rains and dirty streets. The test area having rough streets, however, had much smaller washoff rates

Figure 9.19 Residual Plots for Particulate Residue Washoff Model (all tests combined)



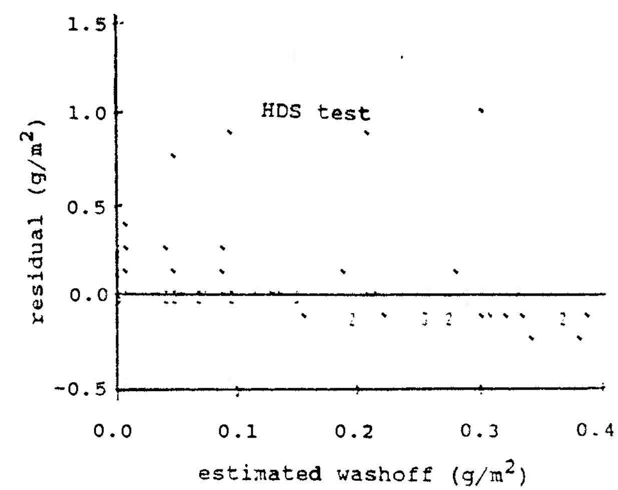
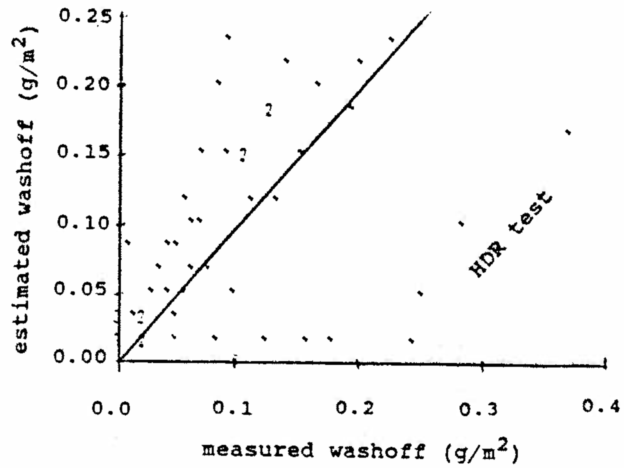
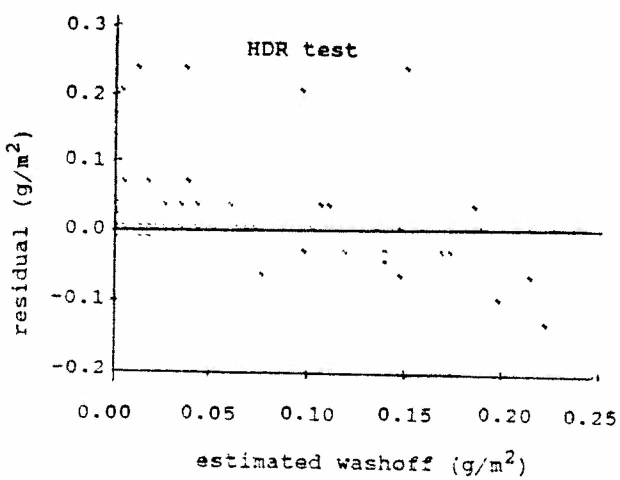


Figure 9.20 Residual Plots for Particulate Residue Washoff Model (all tests combined, except HDS)



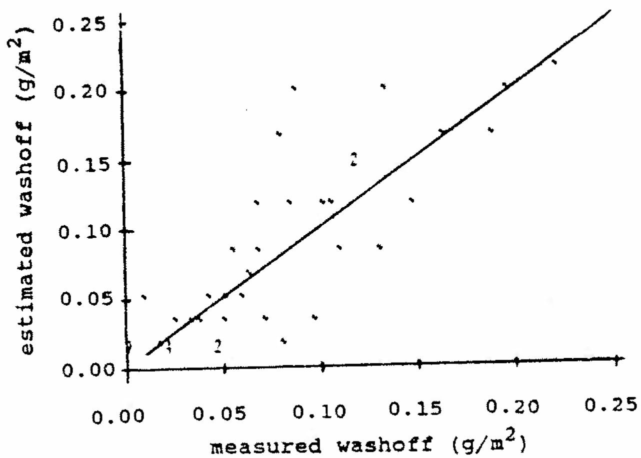


than for the comparable test conducted on smooth streets. Obviously, similar "outlier" analyses can be inappropriately continued until all but one test is eliminated. The last combination model includes all tests, except for high intensity rains and dirty streets. The residual plots shown on Figure 9.21 do not indicate any additional unusual washoff behavior. It is therefore concluded that particulate washoff should be divided into two main categories of models, one for high intensity rains with dirty streets (possibly subdivided according to street texture) and another category for all other conditions.

F. Comparison of Particulate Residue Washoff Using Previous Washoff Models and Revised Washoff Model

This subsection briefly compares the washoff observations obtained during this research with predicted washoff values obtained using the Sartor and Boyd (1972) washoff model (with and without the "availability" factor). Table 9.11 shows the predicted washoff values along with the observed values for the conditions that occurred during the washoff tests. In

Figure 9.21 Residual Plots for Particulate Residue Washoff Model (all tests combined, except for high intensity rains on dirty streets)



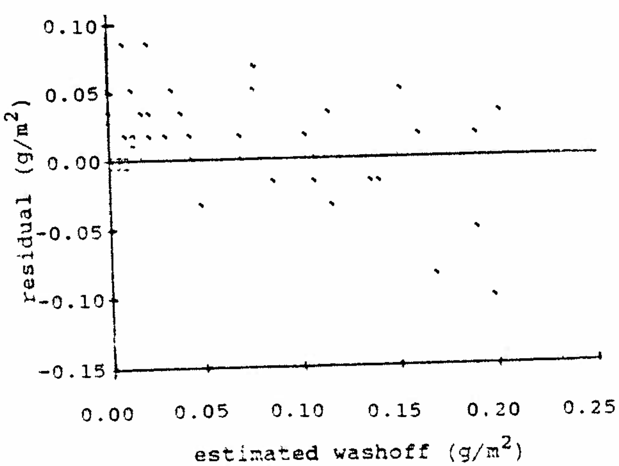


Table 9.11 Comparison of Previous Particulate Washoff Procedures with Observed Washoff

	Calculated Sartor and Boyd Washoff (g/m²)	Calculated Sartor and Boyd Washoff With Avail. Factor (g/m²)	Observed Washoff (g/m²)
Clean Streets			
Light rains	1.47	0.28	0.08 to 0.18
Heavy rains	2.17	1.41	0.28 to 0.45
Dirty Streets			
Light rains	7.73	1.47	0.28
Heavy rains	11.42	7.42	0.30 to 1.5

all cases, serious over-predictions in street dirt washoff resulted by using these common washoff models. Even with the availability factor, the predicted Sartor and Boyd washoff quantities were almost two to more than five times greater than observed. Without the availability factor, the modeled washoff quantities were at least five times greater than the observed values. The residuals (all reflecting over-predictions) of these modeled estimates ranged from 0.2 to 7 g/m^2 when using the availability factor, compared to residuals mostly less than 0.05 g/m^2 when the model developed during this research was used. Lower residuals obtained by using the revised model could be expected because these data were not independent from the data used in developing the revised washoff model.

As stated previously, over-predicted street dirt washoff quantities would result in under-predictions of particulate residue from other sources during model calibration. These over-predictions, especially combined with commonly over-predicted runoff flow volumes (as discussed in Section 7), dramatically affect the relative importance of different urban runoff pollutant source areas and estimated effectiveness of source area controls.

G. Summary of Street Particulate Washoff Tests

This section summarized the street particulate washoff observations obtained during this research, along with the associated street dirt accumulation measurements. The objectives of these tests were to identify the significant rain and street factors affecting particulate washoff and to develop appropriate washoff models. These tests and calculations were also used to clarify apparent confusion caused by misuse of washoff equations in urban runoff models.

The controlled washoff experiments identified important relationships between "available" and "total" particulate loadings and the significant effects of the test variables on the washoff model parameters. Past modeling efforts have typically ignored or misused this relationship to inaccurately predict the importance of street particulate washoff. The available loadings were almost completely washed off streets during rains of about 25 mm (as previously assumed). However, the fraction of the total loading that was available was at most only 20 percent of the total loading, and averaged only 10 percent, with resultant actual washoffs of only about 9 percent of the total loadings. Based on extrapolating the washoff models, only very large rains (possibly approaching 100 mm in depth) could ever be expected to wash off most of the total particulate street dirt load. These very large rains are well beyond the range of any washoff tests. However, observed street dirt washoff during actual rains near this size have not produced substantially greater washoff quantities than observed during the tests conducted during this research. The correctly used exponential washoff models only appear to be applicable for rains in the range of about 3 to 30 mm, which are the most important rains for water quality studies.

The fractions of the particulate residue loadings that were available for washoff was affected by both rain intensity and texture. In many model applications, total initial loading values (as usually measured during field studies) are used in conjunction with model parameters for available loadings, resulting in predicted washoff values that are many times over-predicted. This has the effect of incorrectly assuming greater pollutant contributions originating from

streets and less from other areas during rains. This in turn results in inaccurate estimates of the effectiveness of different source area urban runoff controls.

Accumulation tests were also conducted during this research to determine the street dirt loadings before rains. The industrial street experienced a much greater accumulation rate than the residential street, probably because of increased tracking of debris from unpaved driveways and parking areas and greater deposition of particulates from the heavy car and truck traffic. The smooth streets had much lower initial loadings immediately after street cleaning, but street texture did not affect particulate accumulations as much as land use.

These accumulation and washoff relationships were included in the Source Loading and Management Model to describe street dirt washoff processes. The next section summarizes the use of the complete model to predict warm weather pollutant discharges (for comparison with observed outfall conditions as a verification process) and then uses the model to examine sources of pollutants and the relative effectiveness of different control practices for different land uses.

SECTION 10

VERIFICATION OF QUALITY COMPONENTS OF SLAMM AND ITS USE TO ESTIMATE THE EFFECTIVENESS OF URBAN RUNOFF CONTROLS

A. Introduction

The objective of this section is to demonstrate how the results of this research can be used. The research topics were selected because of apparent inabilities of current models to supply the needed information to decision makers.

This section shows how the Source Loading and Management Model (SLAMM), incorporating the previously described runoff and particulate washoff models, can be used to identify critical pollutant sources in urban areas. The model results can also be used to develop simple cost-effectiveness plots needed for selecting appropriate control programs.

This section begins by summarizing the verification of the water quality components of the integrated model. The complete set of water quality data is contained in the report prepared for the Ontario Ministry of the Environment (Pitt and McLean 1986).

Earlier sections of this dissertation examined impervious area runoff and particulate washoff processes in detail. These processes have usually been poorly represented in urban runoff quality models because of their reliance on previously developed flood analysis and drainage system design models that were developed with an obvious emphasis for large rains. This emphasis on large rains allowed many simplifications concerning runoff losses. Such simplifications can have dramatic effects on runoff predictions for relatively small rains that are of most interest for water quality studies.

The models described in earlier sections of this dissertation emphasized small storm processes and were integrated into a comprehensive urban runoff model (SLAMM). SLAMM also considers other urban runoff processes of importance, including the effects of source area and outfall controls. Descriptions of these

control practices are summarized in reports prepared for the Ontario Ministry of the Environment and the Wisconsin Department of Natural Resources (Pitt 1985 and 1987).

This section summarizes the verification of the water quality components of SLAMM for the test watersheds and shows examples of how it can be used to direct urban runoff management decisions. The verified model is then used in examples to predict the sources of pollutants for the Toronto industrial and residential/commercial test watersheds. Further use of the model is demonstrated by predicting the effectiveness of different urban runoff control programs for a variety of specific Toronto land use conditions.

SLAMM currently does not consider baseflows or snowmelts which were shown earlier to be significant flow contributors. These important sources must therefore be estimated outside of the model for consideration when estimating mass balances and control program effectiveness.

B. Methodology

Many environmental samples were obtained in the two Toronto watersheds to test the source area mass balance hypothesis presented in Section 3, and the small scale particulate washoff models described in Section 9, for complex urban watersheds. Source area particulate quantity and quality monitoring was conducted to examine the magnitude of potential pollutant loadings at different source areas, to calibrate SLAMM for particulate pollutant concentrations for different source areas, and to obtain initial street dirt loading values needed for the washoff models described in Section 9. Sheetflow samples were obtained from the same locations as the particulate samples, but during rains and snowmelts. The sheetflow samples were used to determine the filterable pollutant contributions from the source areas, as needed by SLAMM, and to confirm the availability of pollutants predicted by the particulate sampling from the streets. Outfall water quality samples were obtained to verify SLAMM when applied to complex urban watersheds, after being calibrated with

the hydrology data (described in Sections 4 through 8) and the source area particulate and sheetflow data.

The complete set of source area and outfall data is presented in the report prepared for the Ontario Ministry of the Environment (Pitt and McLean 1986).

Source Area Particulate Sampling

About 150 particulate samples were obtained from many source areas in each test watershed during dry weather. These particulate sampling locations included pervious areas (bare ground, grass, gardens, paths, unpaved driveways, road shoulders, unpaved parking and storage areas, and railroad right-of-ways) and impervious areas (rooftops, footpaths, parking lots, driveways, sidewalks, and roads). In addition, sediment samples were also obtained from the drainage system and the receiving waters. These samples were divided into nine size ranges (from <37 to >6450 microns) and chemically analyzed for nutrients and heavy metals.

The particulate samples from impervious areas were obtained using vacuuming procedures developed during previous research (Pitt 1979). Particulate samples from pervious areas were obtained by using soft bristled paint brushes to selectively obtain samples on the surface that would be most likely removed during rains. Each sample was composed of at least five subsamples obtained from similar areas which were then composited before analysis.

Source Area Sheetflow Monitoring

About 70 wet weather sheetflow samples were collected during several rains and 95 samples were obtained during two major snowmelt periods from the same general locations as the particulate samples. The sheetflow samples could not represent as many surface and land use conditions as the dry samples because of the concentrated sampling demands required during restricted periods, but did reflect the effects of rain on washoff potential and the filterable portions of the source area pollutant discharges.

The sheetflow samples were obtained from various pervious and impervious surfaces throughout the study basins by using grab sampling techniques. Rooftop samples were collected from roof downspouts, while the other samples were obtained using small hand vacuum pumps.

The chemical analyses conducted on these sheetflow source area samples included residuals, nutrients,

oxygen demanding substances, bacteria, and heavy metals. In addition, periodic screening analyses were also conducted for major ions, dissolved heavy metals, PCBs, phenols, and pesticides.

Test Area Outfall Monitoring

Runoff outfall samples were obtained using conventional automatic flow-weighted samplers (ISCO 2100) at two locations: at the 39-hectare Thistledowns mixed residential and commercial site, and at the 154-hectare Emery industrial site (briefly described in Appendix B). ISCO water level monitors and flow recorders were also used to continuously record flow. More than 95 percent of the stormwater flow was sampled at each location during the sampling period, with 21 warm weather runoff events sampled at Thistledowns and 37 warm weather runoff events sampled at Emery. An additional 40 warm weather and 15 cold weather baseflow samples, along with 33 snowmelt outfall samples were also obtained. Constituents analyzed in all samples included conventional pollutants (residuals, nutrients, oxygen demand, bacteria, heavy metals, and major ions) along with periodic pesticide and organic priority pollutant analyses.

C. Verification of SLAMM Quality Components

The model structure, based on a simple mass balance hypothesis, was presented in Section 3. The model predicts outfall quality and quantity conditions for each of many rains by summing the individual source area responses and the effects of the different source and outfall controls. Sections 5 through 8 presented discussions of the local Toronto urban runoff hydrology responses for homogeneous source areas and how these components can be combined to predict outfall flow conditions. Section 9 presented particulate accumulation and washoff conditions for streets. Additional source area pollutant data was also obtained, as described above. This information was assembled into SLAMM. Additional information concerning the effectiveness of different urban runoff control practices (Pitt 1985 and 1987) has also been included in this model.

The particulate and sheetflow pollutant data obtained from the many source areas was statistically evaluated to identify data groupings corresponding to

different land uses, source areas, and rain depths. Each source area component is represented in the model with both particulate and filterable residue pollutant concentrations. The particulate pollutant (greater tha 0.4 microns) concentrations are related to particulate residue washoff loadings from each source area, while the filterable pollutant (less than 0.4 microns) concentrations are related to the source area runoff volumes.

These pollutant forms are kept separated until combined at the end of the model analysis because of the effects of different source and outfall controls. Infiltration practices (including roof drains discharging to pervious areas, infiltration trenches, and porous pavements) affect runoff volume and both filterable and particulate forms of the pollutants. Sedimentation processes (including wet detention basi and catchbasins), along with street and sewerage cleaning, only affect particulate forms of the pollutants, and do not affect flow volumes or filterable pollutants. Drainage system processes (suc as grass swales, or the flow over pervious areas from impervious source areas) affect the delivery of the flows, and the filterable and particulate pollutants.

These partitioned pollutant loadings are then combined to obtain total pollutant loadings and concentrations at the end of the analysis.

The homogeneous area hydrology models described in Section 8, along with the street dirt accumulation and washoff models described in Section 9, were used in developing SLAMM. The source area filterable and particulate pollutant characteristics were obtained through extensive source area monitoring, as described above.

The complete model was verified for the Toronto test watersheds by using these source area models and pollutant characteristics to calculate flow weighted pollutant concentrations during warm weather events. These concentration predictions were then compared to the outfall water quality data, as presented in Pitt and McLean (1986). Figures 10.1 and 10.2 show the excellent agreement between the observed annual flow-weighted pollutant concentrations and the predicted values for both study areas.

These concentration comparisons comprised an independent verification because the source area samples used to calibrate the model quality components were collected during special small-scale sampling

Figure 10.1 Observed and Modeled Outfall Pollutant Concentrations - Thistledowns (mixed commercial and residential site)

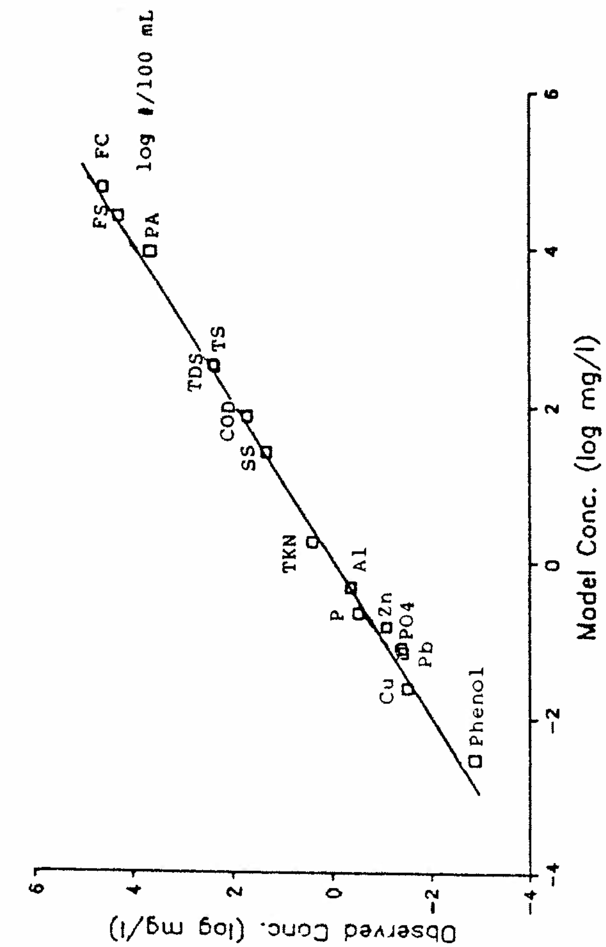
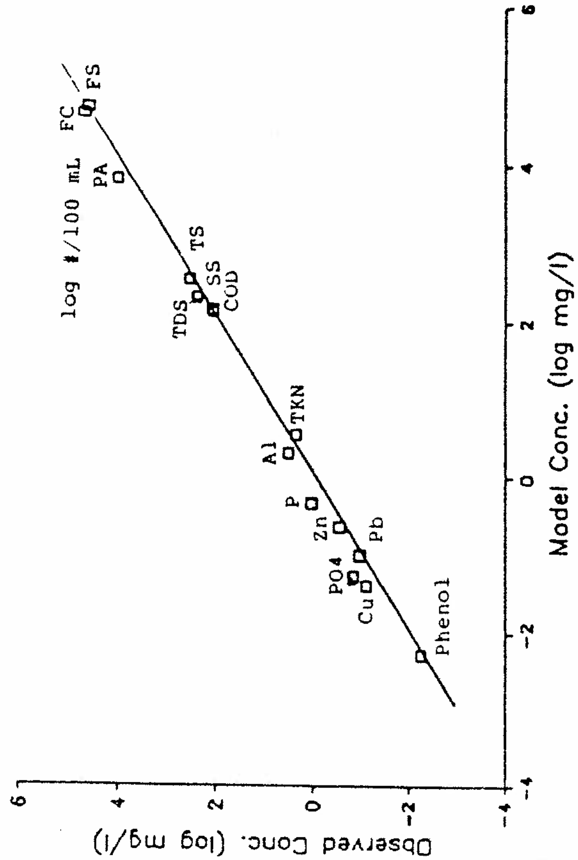


Figure 10.2 Observed and Modeled Outfall Pollutant Concentrations - Emery (industrial site)



programs (most during dry weather or during special simulated rain tests). In contrast, the outfall monitoring was a completely independent large-scale sampling effort for almost all of the rains that occurred. As noted earlier, the flow components of the model were based on both small-scale test and large-scale outfall observations, but were verified using completely independent Milwaukee monitoring results in Section 8.

Annual pollutant mass loadings are obtained by the model by combining the predicted concentrations with the predicted outfall hydraulic responses. The predicted mass loadings are therefore sensitive to specific land development characteristics and source area and outfall runoff controls. The following subsections demonstrate the use of this verified model in identifying pollutant sources and the effects of urban runoff controls for different conditions.

D. Use of SLAMM to Predict Pollutant Sources in Urban Watersheds

One of the most important features of SLAMM is its ability to predict the relative contributions of different source areas for different rains and specific site conditions. This information allows a quick evaluation of the potential effectiveness of different source area urban runoff controls for different site development conditions and rains. As an example, if streets are contributing only 10 percent of a problem pollutant during the critical rains, then street cleaning obviously would not be an appropriate control practice. If many sources contribute the problem pollutant during many of the critical rains, then drainage system controls (such as catchbasin cleaning or the use of grass swales) or outfall controls (such as wet detention or percolation) should be further evaluated.

The calibrated version of SLAMM for Toronto conditions (Toronto/SLAMM) was used to predict the relative source area contributions of different pollutants. Figures 10.3 and 10.4 show example plots of the predicted relative contributions of total residue

Figure 10.3 Total Residue Source Area Contributions - Thistledowns

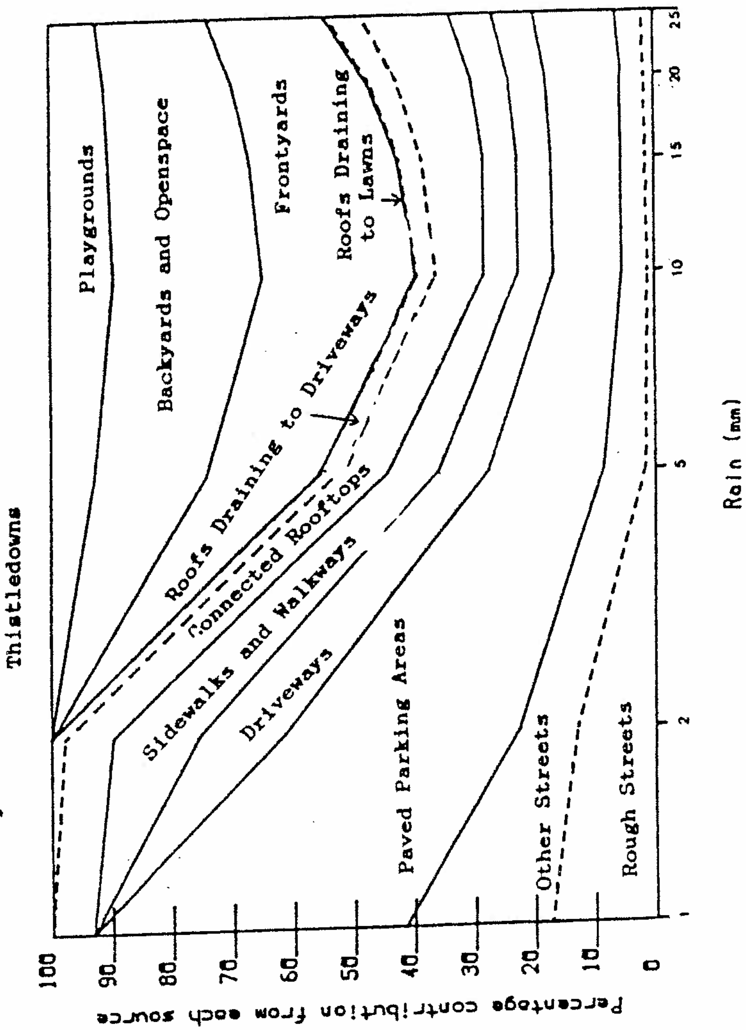
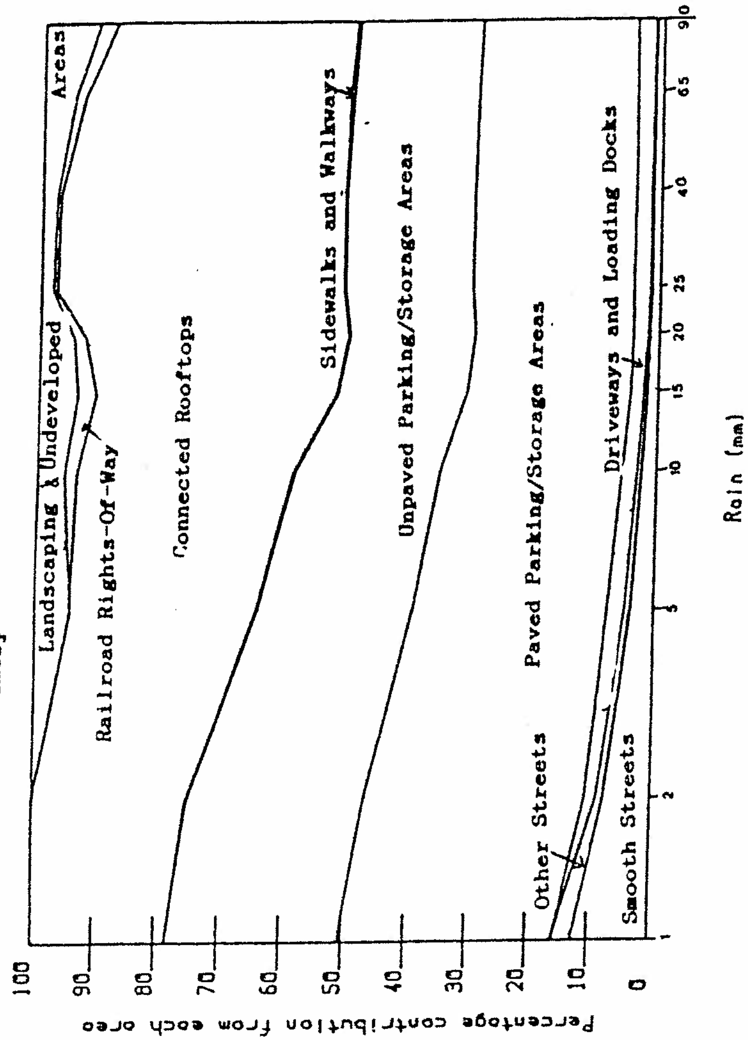


Figure 10.4 Total Residue Source Area Contributions - Emery



from different sources as a function of rain volume. For very small rains, the impervious areas contributed all of the residue. Pervious areas started to contribute important residue (due to erosion) during moderate rain events (greater than about 5 mm).

These analyses were also conducted for other pollutants (shown in Pitt and McLean 1986) and clearly showed the significant effects of the different land uses on relative source contributions. Parking and storage areas were predicted to contribute most of the particulate pollutants from the industrial catchment. For many constituents, paved parking areas and connected roofs also contributed most of the other pollutants discharged in the industrial area. For small events, paved surfaces near the drainage system contributed most of the particulate pollutants from the residential/commercial catchment, while landscaped and open space areas contributed more particulate pollutants for the large events.

E. Potential Effective Urban Runoff Control Programs for Various Land Uses

One of the main functions of SLAMM is to predict the effectiveness of various source area and outfall control options in warm weather stormwater runoff. The effects of the different controls vary greatly, depending on the land uses. Land development characteristics for different land uses were therefore determined during an inventory process that examined about 85 homogeneous areas in the Humber River watershed (Pitt 1985). The most important land development characteristics affecting urban runoff quality and quantity were determined to be land use (industrial or other), directly connected impervious areas, the presence of grass swales, and the nature of the roof downspout connections. This information was used with Toronto/SLAMM to examine the effectiveness of different source area and outfall controls for different homogeneous land uses. The recommendations for control practices for the various land uses found in the Humber River watershed are briefly discussed in the following paragraphs.

Residential Land Uses

Street cleaning in most residential areas may cause significant reductions in the loads of phosphorus, fecal coliforms, and, to a lesser extent, lead at the outfall, compared to no cleaning. Only minor improvements may occur if the frequency is increased beyond current levels, however. It may be difficult to justify increasing street cleaning beyond approximately one pass every month or every two weeks. Extensive spring cleanup and fall leaf removal are expected to be very important.

If roof runoff is not currently directed away from building foundations, walkways, and driveways, then a retro-fitting program to redirect this runoff can be very cost effective. High rise apartments have large paved parking areas; infiltration of the runoff from these paved areas would significantly reduce the flow and load of many pollutants.

The most practical runoff control for lower density residential areas is grass swales instead of concrete curbs and gutters. Grass swales have been shown in monitoring programs to be as much as 90 percent effective in reducing flows and pollutant loads. If grass swales currently exist in an area, changing to curbs and gutters should be strongly discouraged. Groundwater contamination from grass swale infiltration in residential areas is not expected to be significant.

Institutional Land Uses

Street cleaning benefits in school and hospital institutional areas would be similar to those previously described for residential areas. The current levels of street cleaning are important, but increases beyond bi-weekly cleaning may not be justified.

These land uses have large areas of parking lots, paved playing areas, and connected roofs. Redirection of runoff flows from these areas to pervious areas would encourage infiltration. Redirection would also produce significant reductions in runoff volume and loads of most pollutants. Grass swales are also applicable for many institutional land uses and can be very effective.

Commercial Land Uses

Street cleaning at low levels of effort in strip commercial and office areas is important. However,

increases in frequency beyond current levels may not be worthwhile.

The infiltration of runoff (using subsurface infiltration trenches) from paved parking and roofs is the most effective source area control option for all commercial areas, including shopping centers. Pretreatment of water to be infiltrated is necessary to reduce the potential for groundwater contamination. Grit chambers with oil and grease traps should be the minimum pretreatment required in a commercial setting. Shopping center runoff may best be treated with wet detention basins before infiltration to reduce potential groundwater contamination.

Industrial Land Uses

Some increases in the frequency of street cleaning in industrial areas may reduce the pollutant loads at the outfall. Typical street cleaning frequencies (next to nothing) in industrial areas should be increased to at least once per month.

Infiltration in industrial areas can result in significant runoff improvements, but it must be carefully done to prevent groundwater contamination. The source areas for infiltration should be restricted to paved employee parking areas (not truck parking areas) and roofs in non-manufacturing industrial areas, and should require pretreatment.

Because of the heavily contaminated dry weather baseflows from the industrial area monitored during this research, wet detention basins at the outfalls of industrial parks are strongly encouraged. These basins would produce some reductions in both wet and dry weather pollutant discharges. More importantly, the basins would offer an opportunity to control spills that enter the storm drainage system.

The use of grass swales in industrial areas is not recommended because they may contribute to the contamination of groundwater by the heavily contaminated runoff flows.

Open Space Land Uses

Open space areas are relatively unimportant sources of runoff and pollutants. However, important losses through erosion can occur from bare ground or steep hills, especially if they are located near the storm drainage system. Careful evaluations of erosion potential should be made for open space areas, especially if they are undergoing development.

Minimum levels of street cleaning are also necessary for these areas, especially spring cleanup if road de-icing materials were used. Any roof drains should be directed to the large expanses of landscaped land available. Grass swales are quite common in open space areas in the urban Humber River basin and are very effective pollutant controls.

F. Cost Effectiveness of Large Scale Control Applications

Control Options Analyzed

Ten different control programs were evaluated for the complete Humber River urban drainage area. These were made up of various combinations of the source area and outfall controls described above:

1) increased street cleaning,

2) increased street and catchbasin cleaning,

3) large wet detention basins serving 25 percent of the drainage area,

4) increased street cleaning and some large wet detention basins,

5) infiltration of 50 percent of the runoff from residential roofs, high rise residential, commercial, and parts of the industrial roof

and paved parking areas, currently draining to pavement,

6) increased street cleaning and partial infiltration,

7) increased street and catchbasin cleaning and partial infiltration,

8) partial infiltration and some large wet detention basins,

9) increased street cleaning, partial infiltration, and some large wet detention basins, and

10) increased street and catchbasin cleaning, partial infiltration, and some large wet detention basins.

These are all retro-fitted controls (or public works' controls that can be applied in existing areas) and do not include any controls that could only be reasonably installed at the time of construction (such as grass swales). Many of these control options are only partially utilized because of the potential difficulty of installing the practices in all source areas of concern. As an example, outfall wet detention basins are difficult to install in established urban areas. This analysis therefore only assumed that about 25 percent of the outfalls would have suitable areas for these basins. Similarly, redirecting roof drains to pervious areas can only occur in those areas having suitable pervious areas close to the existing drain

locations. Only 50 percent of all currently connected roofs and large paved areas were expected to be able to be diverted. The effects of these source area and outfall controls were calculated using Toronto/SLAMM for the complete urban Humber River basin, and are summarized in the following paragraphs.

<u>Costs of Alternative Control Programs</u>

In order to help select the most appropriate control program, as much information as possible concerning the benefits and problems associated with each complete control program is needed. The Manual of Practice for the Design of Urban Runoff Control Practices (Pitt 1985 and 1987) discusses each individual control in detail and can be very important when final selection of project locations and designs are made.

A multi-objective decision analysis procedure (such as described by Keeney and Raffia 1974) should be used when selecting the appropriate control program. In order to use this decision analysis procedure, the objectives of concern must be identified and the ability of each alternative control program to meet each objective must be known. After control performance, cost is the most obvious objective. Costs need to include both initial capital costs and operation and maintenance costs. Other considerations that may affect the selection of a control program include political feasibility, recreational and educational benefits, aesthetics, safety, and nuisance potential. The Manual of Practice summarizes many of these considerations for the different controls, including how specific design specifications can be used to minimize the adverse characteristics of the control options.

It was beyond the scope of this research to identify the relative importance of these potential objectives (tradeoff functions) for the Toronto area decision makers. However, it is relatively straightforward to produce a simple cost-effective relationship. This relationship, and the associated total alternative costs, will probably be the most important decision consideration. This discussion briefly summarizes cost estimates used in developing an estimated cost-effective relationship for the ten alternative control programs for the urban Humber River catchment. The cost estimates are expected to be sufficiently accurate for these analyses, but absolute

costs for specific Toronto conditions can be expected to be different. These costs (given here in 1986 Canadian dollars) are from the discussions in the Manual of Practice and the specific references are not repeated here.

Street cleaning costs are estimated to be approximately $50 per curb-km cleaned. This cost estimate includes all associated street cleaning program costs including equipment amortization, equipment operating expenses, equipment repairs, labor, overhead, and debris disposal. Based on the measured street density for each land use in the urban Humber River catchment and the increased efforts of the alternative programs, approximately 16,000 additional curb-km of streets would be cleaned each year. The total annual increased street cleaning cost for this control alternative is therefore estimated to be approximately $800,000.

Catchbasin cleaning costs are estimated to be approximately $50 per catchbasin cleaned. This cost estimate also includes all associated catchbasin cleaning program costs. The estimated catchbasin density for each land use in the urban Humber River catchment and the increased effort would result in approximately 60,000 more catchbasin cleanings per year for this component of an alternative program. The increased catchbasin cleaning effort is therefore estimated to cost approximately $3,000,000 per year.

Infiltration program costs are divided into two parts:

1) redirecting runoff from residential roofs currently draining onto pavements, and

2) infiltrating the runoff from large paved parking areas and roofs in high rise, commercial, and industrial areas.

Approximately 20 percent of the estimated 85,000 residential roofs in the urban Humber River watershed drain to pavement. The total cost for this infiltration component is approximately $1,100,000, assuming that only one half of the roof drains can be redirected to pervious areas at a cost of approximately $125 per house.

It is estimated that infiltration trenches capable of completely infiltrating most runoff events would cost approximately $40,000 per hectare of paved area or roof. There are approximately 2050 hectares of high rise, industrial, and commercial roofs and paved parking areas in the urban Humber River catchment. These infiltration costs would be approximately

$41,000,000, assuming that only half of the areas would be suitable for infiltration. Total infiltration program costs would therefore be approximately $42,100,000. Annual maintenance would be minimal, but the life of these infiltration devices may be limited to less than 20 years before reconstruction may be needed.

Initial construction costs of large wet detention basins capable of removing approximately 90 percent of the particulate residue in runoff from all land uses are expected to be approximately $200,000 per hectare of pond surface. About 38 hectares total of wet basin surface area would be needed to treat approximately 25 percent of the land area in the urban Humber River catchment. This is approximately 1.1 percent of the area served. Total construction costs for these wet detention basins is therefore estimated to be approximately $7,600,000. Annual maintenance costs are estimated to be approximately 4 percent of the initial construction costs, or approximately $300,000 per year.

Table 10.1 summarizes the total initial capital costs, amortized capital costs, annual operating and maintenance costs, and total annual costs for the ten alternative control programs. The total annual costs

Table 10.1 Costs of Urban Runoff Control Programs

Program Description	Capital Cost	Annualized Capital Cost (note 1)	Annual Operating and Maint. Cost	Total Annualized Cost total	Total Annualized Cost $/ha
1. Increased street cleaning	note 2	note 2	800,000	800,000	60
2. Street and catchbasin cleaning	note 2	note 2	3,800,000	3,800,000	270
3. Wet detention basins	7,600,000	840,000	300,000	1,100,000	80
4. Street cleaning and detention	7,600,000	840,000	1,100,000	1,900,000	140
5. Infiltration	42,000,000	4,600,000	low	4,600,000	330
6. Street cleaning and infiltration	42,000,000	4,600,000	800,000	5,400,000	390
7. Street and catchbasin cleaning and infilt.	42,000,000	4,600,000	3,800,000	8,400,000	600
8. Infiltration and detention	50,000,000	5,400,000	300,000	5,700,000	410
9. Street cleaning, infilt., and detention	50,000,000	5,400,000	1,100,000	6,500,000	460
10. Street and catchbasin cleaning, infilt., and detention	50,000,000	5,400,000	4,100,000	9,500,000	680

note 1: A loan period of 20 years and an interest rate of 9.5% was assured.

note 2: Street and catchbasin cleaning capital costs are included in the unit annual rate used.

are also given on a unit area basis. The annual costs for the alternative programs range from $60 to $680 per hectare for the complete study area. The capital costs are amortized assuming 9.5 percent interest over 20 years.

Cost-Effectiveness Evaluation

The control program effectiveness and cost data described above were used to prepare a simple evaluation of cost/performance for the ten alternative control programs. Figures 10.5 through 10.7 graphically show total annual costs versus percent pollutant reductions for particulate residue, phosphorus, and lead.

Each control program affects the various pollutants and flow volume differently. For example, only infiltration controls affect flow volume and pollutants mostly in filtrate (soluble or dissolved) forms, such as total residue, filtrate residue, phenols, and bacteria. Less expensive wet detention basin controls affect only those pollutants associated with particulate (nonfilterable or suspended) solids, such as particulate residue, phosphorus, total Kjeldahl nitrogen, chemical oxygen demand, copper, lead, and

Figure 10.5 Particulate Residue Removals for Candidate Control Programs

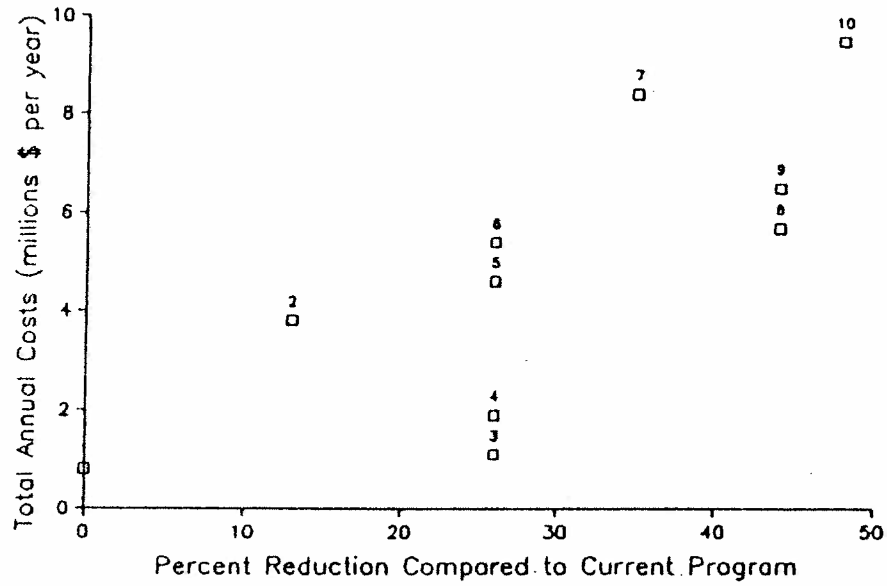


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Figure 10.6 Phosphorus Removals for Candidate Control Programs

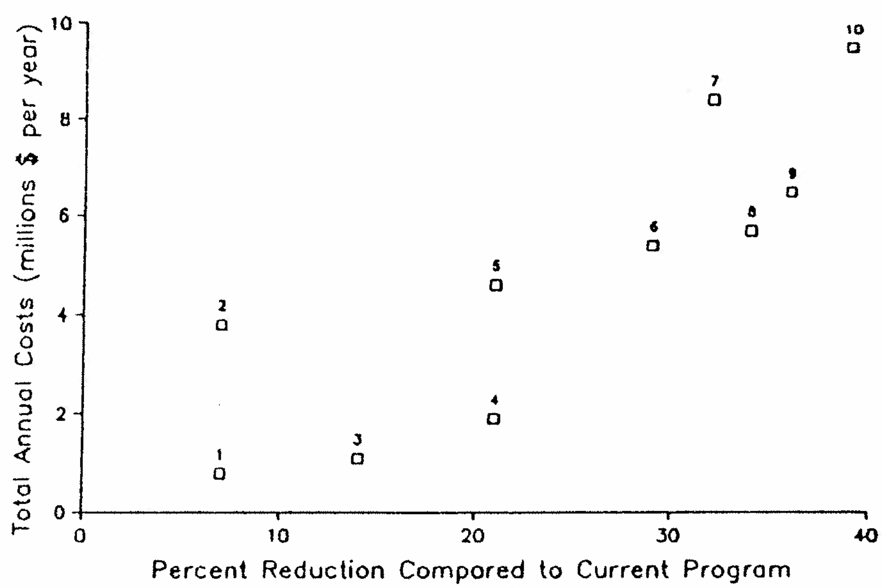
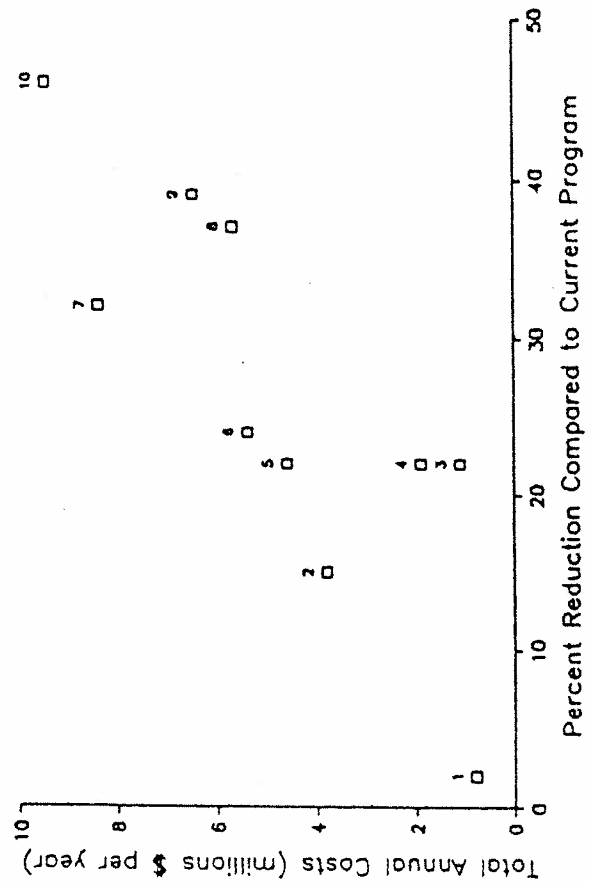


Figure 10.7 Lead Removals for Candidate Control Programs



zinc. A combination of controls is therefore most suitable in order to remove a significant amount of a variety of potential problem pollutants at the lowest cost.

When Figure 10.5 is examined, only three of the ten programs are found to be "cost-effective" (programs 3, 8, and 10) for particulate residue. Program #2 (increased street and catchbasin cleaning) only removes about 13 percent of the particulate residue, but at a cost of almost $4 million per year. Program #3 (detention basins) can remove much more particulate residue (about 26 percent) at a much lower cost (about one million dollars per year). Therefore, program #2 cannot be justified for this situation. Similar observations can be made concerning programs #5 and #6. These programs are much more costly than program #3 for similar reductions of particulate residue load. Program #7 (street and catchbasin cleaning plus infiltration) is also much more expensive than program #8 (infiltration and detention) and results in a smaller reduction of particulate residue. Program #10 includes all of the individual elements (street and catchbasin cleaning, infiltration, and detention), resulting in the highest costs, but is needed if the largest particulate residue removals (about 47 percent) are needed.

The three cost-effective programs for particulate residue therefore include program #3 at $1 million per year and 26 percent control, program #8 at $6 million per year and 44 percent control, and program #10 at $10 million per year and 47 percent control. If 26 percent, or less, control is only needed, then program #3 would be the least costly. However, if control levels between 26 and 44 percent are needed, then program #8 may be the choice. However, it costs about six times as much to achieve twice the control level when comparing programs #3 and #8. Unless the extra level of control was needed (from 44 to 47 percent), it would be hard to justify program #10 which costs almost twice as much for only a very small increase in performance.

When total Kjeldahl nitrogen, phosphorus, COD, copper, and zinc "cost-effectiveness" plots were examined, it was clear that program #8 (infiltration and detention) allows much more pollutant removal to be obtained at a relatively low unit cost as compared to the other control programs. If flow, total residue, filtrate residue, and bacteria are the most important constituents, then program #5 (infiltration alone) is

the most cost-effective solution. However, in order to obtain significant bacteria reductions, it may be necessary to use disinfection in conjunction with wet detention. The most general recommended control program is therefore program #8 (infiltration and wet detention).

G. Summary of Verification and Use of SLAMM Quality Components

This section showed the excellent agreement between predicted and observed outfall pollutant concentrations obtained by using the calibrated version of Toronto/SLAMM. Previous sections of this dissertation presented the verification of the hydrology components of the complete model. This model was used to illustrate how the importance of different source areas varies with different pollutants, land uses, and rains. In most cases, directly connected impervious areas contribute most of the runoff volume and pollutants during small rains (or during the initial portions of larger rains). However, pervious areas can contribute significant portions of many pollutants during larger rains.

The model was also used to illustrate how it can be used to assist in the selection of retro-fitted urban runoff controls for different development conditions. A combination of infiltration and sedimentation controls was found to offer the greatest ability to control a variety of pollutants, at an annual cost of about $410 per hectare per year. If these controls are installed at the time of development, the costs are expected to be much less.

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APPENDIX A

BRIEF HISTORY OF URBAN RUNOFF INVESTIGATIONS

A. Early Urban Runoff Studies

One of the earliest published urban runoff studies was conducted in Moscow, USSR, in 1936 (Shigorin 1956). Shigorin also reported on an investigation that examined cobblestone street runoff in Leningrad from 1948 to 1950. An early study was conducted in Stockholm, Sweden (Akerlinch 1950) from 1945 to 1948. Storm runoff from summer storms was collected from streets and parks, and revealed the potential for shock loadings due to highly polluted individual samples. One of the earliest US studies was conducted in Detroit in 1949 (Palmer 1950). This study included sampling catchbasins that were found to contain standing water that was much more polluted than the outfall runoff water. An English urban runoff study conducted in Oxney in 1954 (Wilkinson 1954?) found that the discharge yields of many constituents increased with longer antecedent dry periods. Street gutter flows were sampled near Green Lake in Seattle in 1959 and 1960 (Sylvester 1960) during a project to evaluate lake eutrophication. It was recommended that urban runoff not be allowed to enter the lake. A study in 1961 in Pretoria, South Africa (Stander 1961), found that commercial area runoff was slightly better than urban runoff from residential areas.

During the 1960s, a series of projects were conducted in the US that examined urban runoff in a much more comprehensive manner than the above mentioned projects. Weibel et al. (1964) conducted an urban runoff study in Cincinnati between 1962 and 1964 in a mixed residential and commercial area. They compared urban runoff to sanitary wastewater and concluded that urban runoff suspended solids was comparable to raw sanitary wastewater, while BOD_5 and COD in urban runoff were comparable to secondary treated sanitary effluent. AVCO Corporation (1970) conducted a study in Tulsa, Oklahoma, during the fall of 1968 to relate urban runoff quality to various land-use characteristics. They concluded that the major source of urban runoff pollution was from material deposited on impervious land surfaces and from drainage channel erosion. Bryan

(1970) conducted a study in Durham, North Carolina, during 1969 and 1970 to also examine the effects of different land-uses on runoff quality.

B. Evaluating Urban Runoff Controls

The information obtained during the above projects was felt to be sufficient justification to begin studies on the abatement of urban runoff. The Storm and Combined Sewer Section of the US Environmental Protection Agency funded a series of projects examining treatment systems for combined sewer overflows (CSOs) and urban runoff during the early 1970s. The numerous pilot-scale and full-scale demonstration projects have been summarized by Lager et al. (1974 and 1977).

Studies investigating the role of street surface contaminants and street cleaning as an urban runoff control option were also conducted during the late 1960s and early 1970s. The US Naval Radiological Defense Lab (NRDL) studied street cleaning as a method of removing fall-out from contaminated areas (Clark and Cobbin 1963). These early NRDL street cleaning studies resulted in effectiveness data that encouraged the use of street cleaning equipment to control urban runoff. The American Public Works Association studied street litter loadings and quality in Chicago during the summer of 1967 for the Federal Water Pollution Control Administration (APWA 1969). Sartor and Boyd (1972) studied the strength of street dirt and the ability of street cleaning equipment to remove these contaminating particulates in ten cities throughout the US from 1969 through 1971. They found very high concentrations of heavy metals in the smallest particle sizes, and related street dirt chemical quality to land-use. They also conducted a series of washoff tests examining the ability of different rains to remove street dirt. Pitt and Amy (1973) made additional chemical analyses of the samples collected earlier by Sartor and Boyd, stressing toxic constituents.

C. Development of Urban Runoff Models

During the early 1970s, several urban runoff models were developed using the available information from the projects mentioned above and various theoretical relationships. These models were developed

as tools to estimate urban discharges from various areas and to estimate the effectiveness of control measures (especially street cleaning and detention basins) in reducing these loadings. The Stormwater Management Model (SWMM) was developed by the University of Florida, Metcalf and Eddy, and Water Resources Engineers for the USEPA in 1971 (Huber and Heaney 1981). The early version was a single event model and was complicated to use and verify. A simplified version for continuous simulations is currently available (SWMM II). The Storage, Treatment, and Overflow Model (STORM) was developed for the US Corps of Engineers by Water Resources Engineers in 1975 (COE). STORM is less complex than SWMM to use, but is usually limited to small, urbanized catchments. Hydrocomp has developed several models, based on the Stanford Watershed Model. HSPF is the most complete version, and was prepared for the USEPA (Donigian and Crawford 1976). Detailed watershed and drainage information is required to operate HSPF.

D. Areawide Planning (208) Studies

During the late 1970s, Areawide Planning Studies ("208" studies) were conducted in about 100 US locations in response to Section 208 of the 1972 Water Pollution Control Act (PL 92-500). These studies were conducted to evaluate the effect of urban runoff on receiving waters, especially those waters that also received treated municipal and industrial discharges. These studies were to be an incentive to local governments to develop their own water quality plans, with minimal federal input. A major effort by the USEPA was made in compiling a large, three volume set titled "Areawide Assessment Procedures Manual" (EPA 1976) detailing how the studies could be conducted. These manuals stressed the use of the urban runoff models previously prepared. Unfortunately, the 208 studies were conducted during a short time period with limited success. Urban runoff discharges and control measure effectiveness were estimated using the above mentioned urban runoff models, with very little monitoring to calibrate and verify the models locally, or to demonstrate the potential success of the recommended control measures. The field personnel did not have

enough experience using the monitoring equipment and much of the newly developed runoff sampling equipment was not sufficiently tested. Therefore, many projects were never able to complete reliable monitoring installations within the short time periods available for the studies. The local recommendations included in the individual "208" plans were typically "laundry lists", including every control measure that could be identified. Street cleaning was high on almost all lists because of the encouraging street cleaning data collected in earlier studies and the high costs associated with the better documented "end-of-pipe" controls (such as detention ponds).

E. Estimated Urban Runoff Control Costs

Nationwide cost estimates to control urban runoff have been periodically made since the mid 1970s. The USEPA Needs Survey (EPA 1975) included urban runoff control costs at an estimated initial capital cost of more than $260 billion. The National Commission on Water Quality (Black, Crow, Eidsness et al. 1975) estimated an initial capital cost of about $290 billion to control urban runoff. Later, estimates made by Heaney et al. (1979) were less (about $100 billion initial capital costs and about $5 billion per year operation and maintenance costs) due to eliminating the cost for storm drainage construction as an urban runoff expense, applying some of the street cleaning costs to aesthetic objectives, and using a continuous distribution of storms for analysis, instead of a single "design storm". These cost estimates were based on a control objective of about 85 percent removal of BOD_5 and were made using urban runoff models (usually STORM) to predict outfall yields and controls for a large number of US cities. Costs to control other pollutants, especially toxic pollutants, were not usually estimated.

F. Street Cleaning Demonstration Projects

Decision makers felt more technical information was needed before expenditures of such magnitudes could be justified. Actual demonstrations of street cleaning effectiveness in reducing urban runoff discharges were funded by the Storm and Combined Sewer Section of the

USEPA from 1976 through 1983. A street cleaning demonstration project was conducted by Pitt (1979) in San Jose, California, from 1976 to 1978. This project studied several types of street cleaners under a wide variety of conditions, and monitored urban runoff simultaneously in two study areas. The street cleaning equipment was operated on actual streets in large drainage basins, in contrast to the earlier street cleaning studies that evaluated street cleaning equipment in small test strips. Only several urban runoff events were monitored during this study because of severe drought conditions. Removals of up to about 50 percent of the runoff total solids and heavy metals were estimated for very frequent cleaning (once or twice per day) on smooth asphalt streets. More common once or twice per month, street cleaning was estimated to remove less than 5 percent of the runoff total solids and heavy metals. Street cleaning was not expected to be effective for removing nutrients or organics, or when operating on rough streets, irrespective of cleaning frequency.

The Water Planning Division of the USEPA funded a street cleaning demonstration project in Castro Valley, California, during 1979 and 1980, as a prototype project for the Nationwide Urban Runoff Program (Pitt and Shawley 1982). This project examined the effectiveness of street cleaning equipment under "real-world" conditions simultaneously with comprehensive urban runoff monitoring. About 90 percent of the total runoff (about 50 runoff events) was monitored during the two years. Frequent street cleaning (about three times per week) was found to control about 35 percent of the urban runoff lead and about 20 percent of the urban runoff total solids.

The Storm and Combined Sewer Section of the USEPA funded a comprehensive street cleaning demonstration study in Bellevue, Washington, from 1980 through 1982 (Pitt 1984). This project was designed to compare the effectiveness of street cleaning in a more humid climate with the earlier data obtained in the much more arid south San Francisco Bay Area. About 95 percent of the total urban runoff was sampled (from more than 350 runoff events) during the two project years. No significant improvements in urban runoff water quality were identified in these Bellevue studies, even for frequent three times per week cleaning. The frequent rains in Bellevue did not allow the streets to become sufficiently dirty for the street cleaning equipment to

be effective. Interevent periods of only three days were common in Bellevue, while interevent periods of 10 to 100 days occurred in the earlier Bay Area tests. Comparisons of the street dirt loading data before and after about 50 storms showed that the washoff of particulates was very size dependent. The rains were only capable of removing the smallest particle sizes (usually less than about 250 microns), while street cleaning was only effective in removing the larger particles (greater than about 200 microns). The rains also left large amounts of small particulates on the streets that were possibly shielded by larger particles. Erosion of adjacent landscaped areas also increased the street dirt loadings of the largest particle sizes.

The street cleaning effectiveness values obtained during these three projects were much less than estimated when using the urban runoff models. The models are suspected of over-estimating the amount of runoff flows from street surfaces at the expense of the runoff flows from pervious areas. The street dirt washoff procedures used in the models are also expected to over-estimate the magnitude of street dirt washoff.

G. Sources and Effects of Urban Runoff Pollutants

Early studies of the effects of urban runoff on receiving waters were mostly restricted to statistically comparing available dissolved oxygen (DO) concentrations obtained during periods of rains, with DO concentrations obtained during dry weather. Keefer et al. (1979) examined available DO data from 104 water quality monitoring sites located throughout the US. They found that DO concentrations during wet weather of less than 5 mg/L were common, and that many stations had greater DO deficits during wet weather than during dry weather. Ketchum (1978), on the other hand, found no relationships between DO and wet weather. Heaney et al. (1980) summarized past studies of receiving water effects and found that well documented cases of receiving water detrimental effects were scarce.

The Storm and Combined Sewer Section of the USEPA funded a study (Pitt and Bozeman 1982) of Coyote Creek, California from 1978 through 1981. This study investigated water and sediment quality, and biological conditions in Coyote Creek as it passed through San Jose. The biological investigations found distinct

differences in the taxonomic composition and relative abundance of the aquatic biota between the nonurban and urban creek sections. The nonurban creek sections supported a comparatively diverse assemblage of aquatic organisms, including an abundance of native fish and numerous benthic macroinvertebrate taxa. The urban creek sections, in contrast, comprised an aquatic community generally lacking in diversity and was dominated by pollution-tolerant organisms, such as mosquitofish and tubificid worms. No one pollutant was expected to cause the observed significant decreases in biological quality in the urban sections of the creek. Very large concentrations of several toxic pollutants, including lead, copper, and zinc, occurred in the urban reach sediments that were present in much lower concentrations in the nonurban creek sediments. Another element of the Coyote Creek project investigated potential sources of urban runoff pollutants. A few dry soil and sheet flow samples from source areas in the south San Francisco Bay Area were analyzed for important urban runoff pollutants. Rain was found to have the lowest concentrations of many of the pollutants, while parking lot and gutter flows had the highest concentrations of most pollutants. Urban soils were found to contain many more pollutants than rural soils. Small storms were thought to be polluted mostly by soils from directly connected impervious areas (streets, parking lots, etc.), while large storms (greater than about 12 mm total rain) would be affected mostly by soils eroding from pervious areas.

Pitt and Bissonnette (1984) summarized the many aspects of urban runoff effects on receiving waters that were recently studied during a four year investigation in Bellevue, Washington. The Civil Engineering Department and the Fisheries Institute of the University of Washington (Pedersen 1981; Perkins 1982; Richey et al. 1981; Richey 1982; and Scott et al. 1982) were funded by the Corvallis Lab of the USEPA to contrast the biological and water quality conditions in urban Kelsey Creek with rural Bear Creek. The urban creek was significantly degraded when compared to the rural creek, but still supported a productive, but limited and unhealthy, salmonid fishery. Many of the fish in the urban creek had respiratory anomalies. The water and sediment quality in the urban creek was not grossly polluted, but flooding caused by urban development had increased dramatically in recent years. These high flows also effectively flushed the urban

runoff toxic pollutants through the small urban creek and into Lake Washington.

These two West Coast studies both showed significant and diverse beneficial use degradations in the urban receiving waters, but the possible causes were quite different. The cause and effect relationships of urban runoff on aquatic organisms is site specific, depending to a great extent on the local hydrologic conditions. In San Jose's Coyote Creek, sedimentation of toxic pollutants in the urban reaches probably were responsible for the aquatic life degradation observed, while the increased flows in the urban Kelsey Creek in Bellevue probably caused many of its problems. The long-term and repeated effects of urban runoff quality and quantity were also found to be more important than the short-term effects associated with specific runoff events. It is very difficult for a receiving water to recover from frequent discharges of pollutants or flashy flows that occur several times a month. Rare events, only occurring once in many years, also have significant effects, but the receiving waters have a much better chance of recovering during the long inter-event periods.

H. Great Lakes Regional Studies

The Great Lakes Water Quality Agreement of 1972 between the US and Canada directed a series of projects to examine the effects of land-use activities on Great Lakes water quality. Task C of this program included detailed surveys of selected US and Canadian watersheds to determine the sources of pollutant discharges to the Great Lakes (agricultural runoff, urban runoff, industrial discharges, sanitary discharges, etc.). The Menomonee River in Milwaukee was studied by the Wisconsin Dept. of Natural Resources, the University of Wisconsin, and Marquette University as an example of an urban/residential watershed undergoing change (Chesters et al. 1979?). A detailed land use survey of the watershed was made, along with monitoring outfalls to the river and tributaries at about 20 locations for water quality. An urban runoff model (LANDRUN) was also developed by Novotny of Marquette and used to extrapolate the measured loadings to other locations in the watershed. The Menomonee River sediments, groundwater inputs, atmospheric inputs, and receiving water biological conditions were also studied. The

study found that near-shore localized water quality problems near major urban centers along each of the Great Lakes may require local urban runoff control. Urban runoff controls would be required for sediment, phosphorus, lead, and other toxicants, and should be directed at critical land-use areas. Point sources should be controlled first, followed by controls at construction and heavy industrial sites. Sewer separation could result in excessive discharges of urban runoff pollutants to the Lakes.

The Ontario Ministry of the Environment and Environment Canada have also sponsored many urban runoff projects in the Great Lakes region and elsewhere in Ontario. The Rideau River Stormwater Management Study in Ottawa was concerned with high bacteria levels at public swimming beaches; it included monitoring of stormwater discharges and Rideau River water quality from 1978 to 1981 (Ontario 1983). Pitt (1983) analyzed the available Ottawa data and collected a series of special source area samples to estimate the sources of the bacteria, their health significance, and potential control. Urban runoff sources, especially dog feces, were expected to be the most significant source of fecal coliforms. In-stream inputs from water birds and gulls on bridges were also expected to be significant bacteria sources. River sediments had very high concentrations of bacteria and easily released large quantities of bacteria when disturbed. Therefore, reduction of urban runoff discharges into the Rideau River may not be rapidly followed by significant pollutant reductions in the water column, or at the beaches. The monitored pathogen populations were not well correlated to the fecal coliform indicator bacteria populations, and direct measurements of the significant pathogens expected (Pseudomonas aeruginosa, Shigella, and Staphylococcus aureus) was recommended. The sources of the bacteria could be better identified by a comprehensive source area (surface sheet flows versus outfall discharges) monitoring of Streptococcus biotypes that are sensitive indicators of human versus other animal feces contamination.

The Toronto Area Watershed Management Strategy Study (TAWMS) is currently being carried out to investigate discharges of problem pollutants (especially bacteria and toxic heavy metals) to the local receiving waters (rivers and Lake Ontario) (Pitt 1983).

I. Nationwide Urban Runoff Program (NURP)

The need to determine the effects, discharges, and controls of urban runoff during comprehensive field programs on a nationwide basis became evident after the "208" studies were completed, and cost estimates for urban runoff control were made. The Water Planning Division of the USEPA funded 28 projects, for about $30 million, during the period of 1979 through 1983, including a comprehensive project in Milwaukee (Bannerman et al. 1983). The final nationwide report has been completed (EPA 1983) and summarizes these projects. These projects all included successful urban runoff characterization components. About 80 monitoring stations were located in residential, commercial, and mixed land-use areas. Only four stations were located in light industrial areas, and no monitoring occurred in medium or heavy industrial areas. Many thousands of constituent concentration observations were made for a wide range of pollutants, including nutrients, heavy metals, solids, and oxygen demand. A special sub-study investigated toxic organics at about 20 locations.

The effectiveness of several types of urban runoff control measures were also monitored. Control measures investigated included detention basins, street cleaning, recharge basins, grass swales, and wetlands. The most effective control measure evaluated was detention basins. Several studies also investigated receiving water impacts of urban runoff, but stressed direct effects that occurred during runoff events. Comprehensive long-term receiving water effects were not monitored.

The Milwaukee NURP project (Bannerman et al. 1983) monitored urban runoff flows and quality at eight sites, including various residential and commercial areas. Many street cleaning effectiveness measurements were made, and a small set of rain washoff observations were obtained. Detailed land-use and meteorological information was also obtained.

The large amounts of data collected during NURP have been entered into the EPA's STORET and the USGS's WATSTORE computer files, and will be available for additional analyses. The individual project reports typically contain much more detailed data analysis than the nationwide effort. The individual reports are

usually available through the National Technical Information Service (NTIS).

APPENDIX B

SITE DESCRIPTION

Figures B.1 and B.2 are maps of the Toronto Area Wastewater Management Study (TAWMS) area and indicate the locations of the two test watersheds monitored during this research. The study areas are within the Humber River watershed which was selected for study because of recognized urban runoff impacts.

The following list summarizes some of the water quality problems currently existing in the Humber River, based on a review of existing data (Pitt 1983).

- o Lead, zinc, and fecal coliform concentrations have very pronounced gradients along the Humber River, indicating significant downstream (urban) sources.
- o Copper and cadmium concentrations frequently exceed the water quality criteria along the whole length of the River.

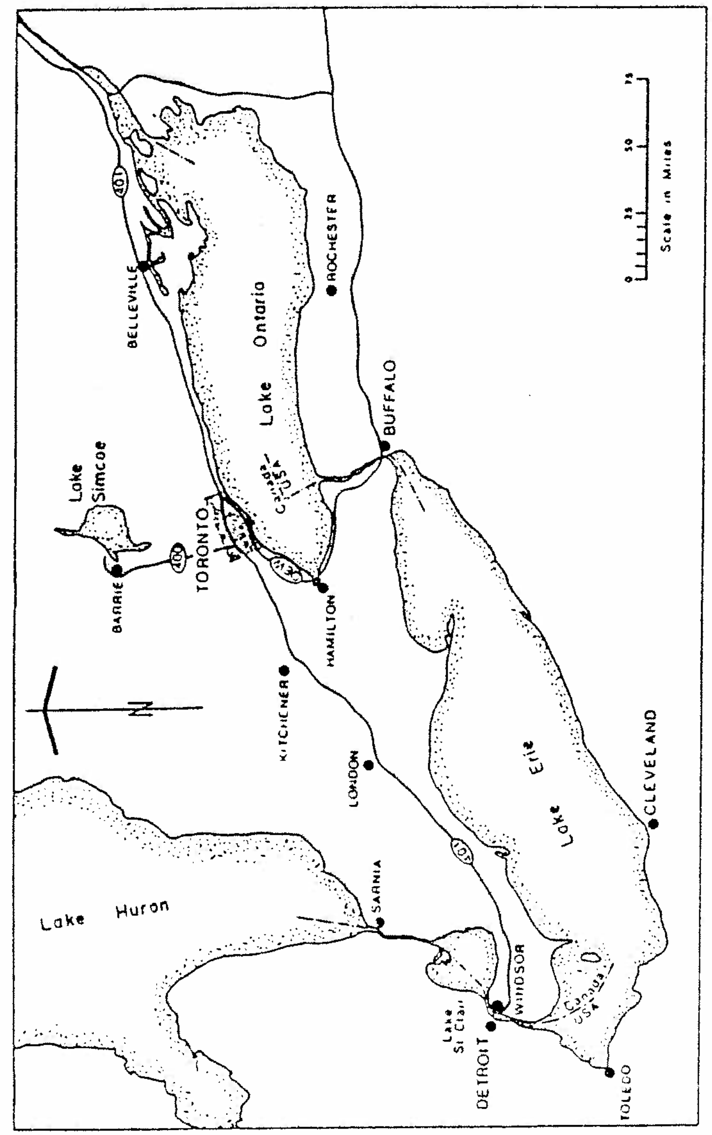
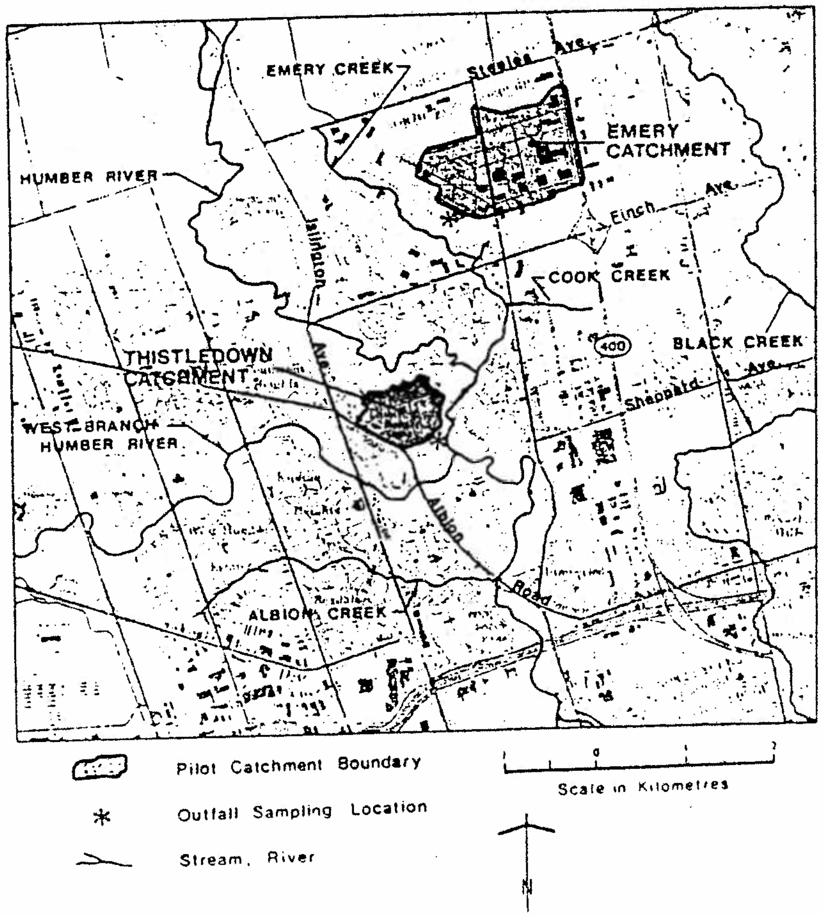


Figure B.1 Map Showing Location of Toronto, Ontario

Figure B.2 Map Showing Location of Test Watersheds in Toronto



o Mercury, chromium, and <u>Pseudomonas aeruginosa</u> are also likely present in problem concentrations.

- o Pentachlorophenol and 2,4-D are the most commonly found toxic organics in the Humber River.
- o The highest Humber River pollutant concentrations occur during the most extreme high and low River flows.

The most common land uses in the urban portion of the Humber River study area (18,500 ha, or 70 square miles), in order of decreasing abundance, include: single family residential areas, parks/open spaces, and industrial areas. Two sites were selected for monitoring in the Humber River watershed: an industrial site in North York, and a mixed residential and commercial site in Etobicoke. The following paragraphs briefly describe these two test watersheds.

A. Thistledown Test Watershed

The Thistledown test watershed covers approximately 39 ha of residential and commercial areas surrounding Thistledown Boulevard in the City of Etobicoke. It is approximately bounded by the Humber River to the east and north, and Albion Road on the southwestern side. Most of the test watershed consists of single family dwellings that are 10 to 20 years old. Table B.1 characterizes the land uses within the Thistledown test watershed.

Table B.1 Thistledown Land Uses

LAND USE	AREA (ha)	AREA (%)
Single family dwellings	29.5	75.9
Multi-family dwellings (townhouses)	2.4	6.3
Shopping center	2.1	5.4
Open space	0.2	0.5
Schools (2)	4.5	10.9
Church	0.4	1.0
Totals	38.9	100.0

Approximately 9 percent of the test watershed area is used for roadways. These roads are generally two lanes wide (one in each direction) with parking allowed, and have a total length of approximately 4.8

km. Most of the roads have smooth to intermediate textures and are in good condition. However, approximately 35 percent of the roads are in moderately poor, or worse, condition.

Approximately 20 percent of the roof drainage is directly connected to the storm sewer system, with the remaining roofs draining to driveways or lawns (40 percent each).

The roadside drainage system is mixed. Approximately 57 percent of the roads have grass swales connected to the storm sewer system by gratings and catchbasins. These swales occur only on the flat eastern half of the test watershed. There are approximately 90 meters of sealed swales and approximately 2000 meters of concrete curbs and gutters forming the other 43 percent of the drainage system. The concrete curbs are located on the steeper grades of the test watershed, which have road slopes of up to about 5 percent. During this study, runoff was frequently observed in the concrete gutters. However, it was rarely observed in the grass swales, even during high intensity thunderstorms.

A shopping center is located on the southwestern boundary of the test watershed, with paved parking areas making up the bulk of this land use (72 percent). A small service station and the loading bay for a supermarket are also located within the parking area. The bulk of the land described as schools consists of grass playgrounds.

B. Emery Test Watershed

The Emery test watershed was selected for study because the Humber River and Tributary Dry Weather Outfall Study (Gartner Lee and Associates 1984) identified this area as one of the most significant contributors of contaminants to the Humber River system.

The Emery test watershed area is approximately 154 ha, located in the City of North York. It is surrounded by Highway 400, Finch Avenue, Islington Avenue, and Steeles Avenue. It is predominantly an industrial area with relatively flat terrain.

The Emery test watershed contains a variety of industrial activities, as described in Table B.2. It contains little heavy industry, such as power plants or steel mills. Most of the industrial activity is

Table B.2 Emery Industrial Activities

INDUSTRIAL GROUP	NUMBER OF BUSINESSES	TOTAL AREA (ha)	AVERAGE AREA (%)	AVERAGE SIZE (ha)
Chemicals	13	20.6	13.5	1.5
Metal dealers and manufacturers	14	10.43	6.8	0.75
Contractors, machinery	5	5.49	3.6	1.1
Printer	3	2.81	1.8	0.9
Utilities	1	1.4	1.0	1.4
Furniture Manufacturing	4	6.86	4.5	1.7
Mixed Industries, Hardware & Bldg. Supplies)	3	2.96	1.9	1.0
Food Industry	11	12.44	8.1	1.1
Offices & Warehouses	17	12.84	8.4	0.75
Vehicle Repair	5	2.04	1.3	0.4
Miscellaneous Manufacturing	9	7.67	5.0	0.85
Electronics	4	30.4	19.8	7.6
Foundries & Welding	3	1.05	0.7	0.35
Metal Plating	2	1.15	0.7	0.57
Waste Dealers	4	8.87	5.8	2.2
Tiles	2	0.71	0.5	0.35
Textiles	2	2.11	1.5	1.1
Glass	2	2.25	1.5	1.1
Totals / Averages	104	153.7	100.0	1.5

categorized as medium industry, i.e. processing goods for final consumption.

The test watershed has 7.3 km of roadways, including two major arterial roads (Signet Drive and Weston Road). Traffic counts of 600 to 800 vehicles per hour are typical on these major roads. Road textures are predominantly smooth and are in moderately good to very good condition. All roads have concrete curbs and concretes or asphalt gutters. On-street parking occurs only on 7 percent of the roads. This test watershed also contains 4.1 km of main line railroad track with several industries having their own railroad spur lines.

APPENDIX C
URBAN HYDROLOGY MODELS

A. Introduction

This appendix summarizes many literature discussions pertaining to the prediction of urban runoff flows, especially for impervious areas. Some general model comparisons are made, but this is not intended to be a comprehensive guide to the selection of runoff models. The objectives of this appendix are to present the variety of approaches currently used in predicting urban runoff and to discuss their benefits and short-comings. These discussions led to the need to develop the research described in this dissertation, specifically, to develop a simplified method to predict runoff from impervious areas applicable for water quality studies.

Satisfactory predictions of sanitary sewer flows are possible because of the relatively stable input conditions. Storm drainage flows are much more difficult to predict because of the uncertain, nonlinear responses created by uneven rain and heterogeneous surface characteristics in many watersheds. Storm drainage designers have therefore relied on simple procedures that result in conservative estimates of peak flows. These procedures result in over-estimated flows for the common small rains that may be of most concern in urban runoff studies.

There are several common methods that have been used to predict flows from impervious areas. The Rational Method is used in the STORM model for impervious area flows (Abbott 1977). SWMM II and HSPF only reduce impervious area flows by an initial detention storage value and assume the rest of the rain as runoff (Bedient et al. 1975; Donigian and Crawford 1976). WURM (Novotny 1983) uses an initial detention storage value of 1.6 mm, based on an limited Chicago study in 1960 (Tholin and Kiefer 1960), for the only runoff losses for impervious areas.

B. Summary

This appendix contains a summary of urban hydrology modeling issues and background information

leading to the need to conduct this dissertation research. This appendix discusses the confusion that commonly occurs when planners try to use an inappropriate urban runoff model that was based on early drainage and flooding methodology for investigating urban runoff water quality issues. Many researchers have previously discussed the problems associated with the basic assumptions of these common modeling procedures and have compared the basic model types. However, little information is available from actual field investigations that can be used to correct the most common problem, mainly, dealing with impervious area runoff losses during common small storms.

Design Objective Basis of Models

Urban hydrology modeling was initially developed as a method to design drainage systems. Unfortunately, many aspects of urban runoff flows are difficult to predict because of uncertain nonlinear responses created by uneven rain and heterogeneous surface characteristics of most urban watersheds. Simple procedures have therefore been used which result in conservative estimates of peak flows, acting as a safety factor in drainage design. When urban runoff water quality models were developed, it was common to base these new models on the older hydrologic models. Unfortunately, these drainage design models contained many assumptions that severely affected runoff predictions for small events that are of most concern in water quality studies.

Model Comparisons

Several model reviewers have compared different urban runoff hydrology models with actual field data and have found that the simpler statistical models commonly outperformed the more complex theoretical models. No matter what model is selected, good calibration and input information is needed. Model selection must be based on the actual identified needs of the user, not on preconceived prejudices concerning the model type (such as is common against "black-box" or statistical models).

Needs for Specific Procedures to Predict Hydrology Responses for Water Quality Models

A model that effectively addresses small events and flow and pollutant contributions from different source areas is needed for water quality models. Urban runoff water quality studies are most concerned with the common small events because they contribute most of the annual pollutant discharges. Urban runoff studies must also consider source area controls. Lumped models (such as the rational formula or the Soil Conservation Service curve number method) that combine the drainage area characteristics into a few parameters may be capable of investigating outfall characteristics or outfall controls, but cannot be used to evaluate the importance of individual source area contributions or the effectiveness of source area controls. Land development characteristics (backyard landscaping, rooftops directly connected to the drainage system, streets, drainage system type, etc.) are usually much more important than the distribution of soil types when determining the variable source areas contributing pollutants and flows. Because of the poor flow loss assumptions concerning small flows common to many existing urban runoff models, an accurate representation of source area contributions and control effectiveness is usually not possible.

Significant Factors Affecting Runoff

Few statistically derived models are based on a wide enough range of field observations, or include enough site information, to identify the important site and rain variables affecting urban runoff processes. An example is the little concern given to identifying the type of drainage system present in test watersheds. Several urban runoff monitoring studies have shown that grass swales can reduce runoff volumes by as much as 90 percent. Large step-wise regression analysis projects have been conducted in an attempt to identify the significant site and rain characteristics affecting runoff without knowing if the drainage areas were served by grass swales or concrete curbs and gutters. As another example, soil characteristics are usually considered to be a significant site characteristic. The site's soil types are usually identified from soil maps representing undisturbed native conditions. Urban areas are characterized by large expanses of grossly

disturbed soils that have little hydrologic resemblance to the native soil descriptions that are used in most modeling studies.

Constant Rv Problems

Another problem with urban hydrology analyses is the common use of constant (and large) runoff coefficients used to estimate runoff from impervious areas. This assumption does not significantly affect runoff predictions during large drainage system design events, but it results in large over-predictions of runoff from these areas during common small events. The use of constant runoff coefficients was found to commonly result in runoff prediction errors greater than 25 percent.

Over-Extensions of Model's Intentions

Many of the simple hydrologic models have also been incorrectly used for drainage design purposes, or extended far beyond what their initial assumptions allow. For example, a common error associated with using the rational method is not using the time of concentration as the critical rain intensity averaging period. Typically, the same rain intensity time increment is used for many different types and sizes of drainages in an area. The rational method also has been overly extended in attempting to predict total storm runoff volumes or to produce runoff hydrographs in many urban runoff models. The rational method is limited to estimating only a single point of the hydrograph, the peak; useful for estimating peak flows for small watersheds and for small rains. It must be used with care when designing drainage systems for large areas (greater than about 40 ha) and for recurrence intervals greater than about 5 years.

The Soil Conservation Service curve number (CN) procedure is increasingly being used for drainage system designs and for water quality studies. Many modelers are concerned about the short-comings of the basic equations used to develop the SCS CN procedure, especially the fixed relationship between initial abstractions and total ultimate losses, and the assumption of zero ultimate infiltration (instead of a small, but steady ultimate infiltration rate). These assumptions may not adversely affect the use of the SCS CN procedure for drainage design studies using severe

storms, but may cause some problems for water quality studies investigating small storms. The selection of a CN value is very critical, and has been demonstrated to vary for different storm volumes at the same site. The observed CN value typically decreases during an event, and is smaller for large events than for small events. If a constant suggested CN value is used, small storm flows may be severely under-estimated (due to the relatively large initial abstractions assumed). Again, the SCS CN procedure has been used in ways for which it was not originally intended. It was originally developed to estimate total storm volume, but attempts have been made to use it to produce hydrographs.

Several unit hydrograph procedures have been statistically developed to overcome problems associated with the rational and SCS CN methods. They were mainly conceived as drainage design tools and not to investigate water quality problems. They do allow estimates of peak flows and total storm volumes to be made, but they do not assume the aspect of "variable contributing areas" that is extremely important in urban areas. Unit hydrographs are difficult to produce from observed data as they require rainfall-runoff observations for constant intensity storms having rain depths between 13 and 45 mm. Very few individually monitored storms satisfy these requirements and complex storms must be decomposed to produce individual "unit" elements. It may be possible to develop a set of standard hydrographs for an area for different rain volumes (to reflect variable contributing areas), but little runoff data for constant intensity storms is available and any decomposition assumptions would contradict the variable contributing area information desired.

Hydrodynamic physical models typically attempt to model an urban area using cascading planes, kinematic wave theory, differential equations of continuity, and an approximation of the momentum equation. The complexity of urban areas requires many simplifying assumptions to solve the equations needed for this approach, with little improvement (if any) in the usefulness or accuracy of the resulting model.

NURP Data Analysis

Urban hydrology observations for many storms at many locations have been recently obtained during the Nationwide Urban Runoff Program (NURP) studies (EPA

1983). The data have demonstrated the importance of variable contributing source areas and the need to use variable runoff coefficients for different rain volumes. These observations and the literature on the short-comings of commonly used models lead to the need to investigate urban hydrology runoff loss processes during small (water quality) events as a major objective of this dissertation research.

C. Types of Models

Linsley (1982), in a paper summarizing urban runoff models, defined a model as a mathematical or physical system obeying certain conditions. The behavior of a model must be analogous to the system under study. Linsley felt that a comprehensive literature search would uncover at least several hundred, if not several thousand, models that have been used to predict runoff from rainfall information. He included in his review paper an interesting set of definitions for the many adjectives that have been used to describe hydraulic models:

"Deterministic-- Based on the assumption that the process can be defined in physical terms without a random component.
Stochastic-- Based on the assumption that the flow at any time is a function of the antecedent flows and a random component.
Conceptual-- Model is designed according to a conceptual understanding of the hydraulic cycle with empirically determined functions to describe the various sub-processes.
Theoretical-- Model is written as a series of mathematical functions describing a theoretical concept of the hydologic cycle.
Black box-- Model uses an appropriate mathematical function or functions which is fitted to the data without regard to the processes it represents.
Continuous-- Model is designed to simulate long periods of time without being reset to the observed data. Such models require some form of moisture storage accounting.
Event-- Designed to simulate a single runoff event given the initial conditions.
Complete-- Includes algorithms for computing the volume of runoff from rainfall and distributing this volume into the form of a hydrograph.
Routing-- Model contains no algorithms for rainfall-runoff but simply distributes a given volume of runoff in time by routing or unit-hydrograph computations.
Simplified-- Uses algorithms which have been deliberately simplified, or uses large time increments to minimize computer running time."

He used these adjectives to describe several different urban hydrology models. As examples, the related Stanford Watershed Model (SWM) and the Hydrocomp Simulation Program (HSP) were described as deterministic, conceptual, continuous, and complete, while the Storm Water Management Model (SWMM) was

described as deterministic, conceptual, event, and routing oriented. Almost all of the models he reviewed were deterministic and most were conceptual.

Freeze (1974) earlier described conceptual and empirical models based on whether (conceptual) or not (empirical) the modeling process is affected by physical processes acting on the input variables. He described the majority of rainfall-runoff models then available as stochastic empirical (containing random variables not affected by physical processes) models, or as "black box" models. He felt that these models satisfied engineering design needs, but "did not provide any insight into the internal mechanisms of the hydrologic cycle". He felt that a "fully-illuminated white box" model was needed (based on a conceptual modeling approach) to better answer basic hydrologic problems. He concluded that the "ultimate" model should be conceptually based, but still have stochastic variables.

These labels may create more confusion than insight. Many relatively simple models not only have numerous descriptions for different model elements, but they also have conflicting descriptions as well. As an example, theoretical process descriptions are commonly coupled with conceptual and statistical (black box) descriptions. This is much more common with water quality models that have been constructed based on older hydraulic models (such as the development of HSPF from HSP from SWM). Each process contained in a model should have its own unique set of descriptors (deterministic or stochastic; and conceptual, theoretical, or black box), while the overall model design also dictates another set of descriptors (continuous or event; plus possibly complete, routing, and simplified). A complete set of descriptors would therefore become very confusing. It would be much better if the processes and the model design were well documented.

Another problem with these descriptors is that some have relatively strong prejudices associated with them. As an example, many modelers feel that theoretical models are the "best" because they should be more transferable, while black box statistical models are only applicable for the site where the data was collected. Obviously, all models need field data for calibration and verification. The understanding of the site and meteorological characteristics under which the field data were obtained, along with the complexity

of the model, should allow a judgment of the applicability of the model at another location. Most modelers and model users would feel comfortable with a model that is mostly conceptually based, indicating that most of the processes being modeled are understood. Many model users also have problems with models that are mostly theoretically based. Theoretical models typically demand extensive input information that may be obscure, difficult, and costly to obtain, forcing the use of general default parameter values.

Troutman (1985a) argued the preconceived differences between deterministic models or black box models. He concluded that the distinction between these two seemingly conflicting categories of models was not at all clear, or important, when analyzing errors. He found that some of the confusion in these model categories was because some users categorized statistical models as black box models (such as defined above by Linsley in 1982). He gives as an example the general assumption of runoff that tends to vary proportionally with rainfall. This conceptual relationship is typically reflected by a very simple statistical black box model. He further shows that many of the most complex physically based conceptual hydrologic models currently used contain many process descriptions where some of the variables are simply statistically related to other variables. Because these models are large and complex, these relationships are commonly overlooked. His major conclusion is that any rainfall-runoff model can be defined as a conceptual model, and that the distinctions between black box and physically based (conceptual) models are not clear or useful. He states that every model becomes a statistical model when the errors are rigorously and objectively examined by representing the errors as random variables having a probabilistic structure.

D. Problems Associated with Current Urban Runoff Models

The "best" model in the world would not be very useful if applied incorrectly, or if it did not address the questions at hand. There are many urban runoff models because there are many different needs. There are also many different review papers of the available models to help a user select the most appropriate model. A model containing many errors and problems according to a comprehensive review article may in fact

be the best model for a specific user if the user understands the model's limitations and the model addresses the user's specific questions. This short discussion summarizes some of the general (and conflicting) problems identified in a selection of model review papers.

Sorooshian and Gupta (1983) found several "problems" in their review of urban runoff modeling. Many models have been developed more as intellectual exercises than as useful tools. There is therefore a need to find a balance between a model's complexity (and cost of obtaining needed input information) and a model's usefulness. The current state of urban runoff modeling is also very fragmented. It is very difficult for a potential model user to select the "right" model because of the seemingly uncoordinated development (and therefore selection) of available models. Most models can also be very difficult to calibrate correctly, especially when using "automatic" optimization procedures. This difficulty is potentially caused by the interdependence of model parameters, very little actual significant sensitivities between apparently significant parameter variables and the model response, and discontinuities and local optima in the resulting sensitivity analysis response surfaces.

Klemes (1983) identified "structural arbitrariness and over specification" as the greatest weakness of current urban runoff models. The ability of a many-parameter model to reconstruct past hydraulic records "can seldom fail" (irrespective of the many literature reports, such as identified above by Sorooshian and Gupta 1983, of many such failures). He was most concerned by the use of unrealistic parameter values (from a conceptual viewpoint) to enable satisfactory calibration. Defective conceptual model structure was compensated for by an excessive number of degrees of freedom in many models. As described earlier, the use of unrealistic parameter values can significantly affect the relative importance of source area contributions and controls in urban runoff models and was a major reason for the research conducted during this dissertation.

Troutman (1985a), however, identified a practical limit to the potential maximum complexity of conceptual models, at least in their ability to approach being truly deterministic. Examples given of this limit are the great diversity of soil conditions which occur over

a study area that would require an almost unlimited number of site sub-divisions, or the use of averaged data to completely describe the infiltration process. He also pointed out that some physical processes will always be less understood than other processes, with the model parameters associated with the less understood processes being less meaningful physically.

Loague and Freeze (1985) also discussed typical model errors, input errors, and parameter errors common to rainfall-runoff modeling (mostly after Lettenmaier 1984). Model errors result in the inability of a rainfall-runoff model to accurately predict runoff, even given the correct estimates and inputs. Input errors are associated with incorrect input rain information and are usually caused by rain gauge measurement errors. Parameter errors can be caused by highly interdependent parameters during calibration, or the above described problem of having to use averaged input parameter values to represent variable parameter conditions over an area.

E. Calibration and Verification Needs for Urban Hydrology Models

Calibration is the process where a model is "adjusted" to obtain a set of responses to a given set of input conditions that are adequately similar to the measured responses. Verification of the calibration process is used to test the adequacy of the model adjustments, also using completely monitored input and output conditions, but for different events than were included in the calibration data set. Loague and Freeze (1985) stated that many <u>favorable</u> model performance reports for event-related models are incorrectly optimistic because of a failure to carefully distinguished between the calibration and verification processes (or data sets).

Sorooshian and Arfi (1982) stated the common opinion that calibration of a model is the most critical stage of the overall modeling process. The determination of the adequacy of model calibration, however, is typically difficult. As in many critical processes, the use of the model should determine how well calibrated the model "needs" to be. Loague and Freeze (1985) described prediction and forecasting as

the two main uses of models. They defined prediction as the process used to develop a set of simulated hydrographs that are used for risk-assessment related engineering design, and forecasting as the development of hydrographs of selected events that are used in making specific operational decisions. They stated that model calibration and verification adequacy can be evaluated statistically for predictive (design) uses, such as by measuring and statistically comparing specific hydrograph characteristics of the series of events studied. Forecasting model use, in contrast, requires comparing each event hydrograph separately which is much more critical of the calibration and verification processes.

Sorooshian et al. (1983) also emphasized that the quality (information contained and collection efficiency) of the calibration data is as important as the quantity of the data used. The calibration data should be representative of the conditions under which the model will be used, but extensive numbers of monitored events used in calibration may not produce the best results. In many cases, it is not even possible to obtain data representing the conditions under evaluation, especially when evaluating alternative design options for a proposed development.

Sorooshian and Gupta (1982) compared manual and automatic calibration procedures. In the manual process, the need for a trained hydrologist (having skill, experience,and intuition) is very important. The model parameters are adjusted subjectively, based on the model results for different conditions. In the automatic calibration procedures, the parameter values are changed automatically during many computer runs based on a mathematical error function. In many cases, a trained hydrologist, knowing the model's structure and processes, can efficiently obtain an adequate calibration. Automatic procedures typically have greater difficulty appreciating the extent to which model users may extrapolate beyond the calibration data limits.

Troutman (1985a) stated that the model calibration and verification processes must satisfy two functions: the model should result in good runoff predictions, and the model parameters should be generally realistic. If the calibrated parameter values are unrealistic, he concluded that the model is physically unrealistic, or that the input data are poor. These potential reasons

must be extensively explored instead of ignoring the calibration results. Guidelines for acceptable parameter values generally take the form of default values (with "acceptable" ranges). These default suggestions have been removed from recent versions of SWMM because of their common abuse (using them instead of collecting local calibration data, or the calibration process not resulting in parameter values within the guidelines).

F. General Comparisons and Selections of Urban Hydrology Models

As stated previously, there exist many literature discussions concerning the relative merits and problems of various urban runoff models. They can be useful sources of information to potential model users who have specific modeling objectives. However, without such objectives, model selection is of little use. Even if model selection is obvious, Linsley (1982) stressed the importance of good data required by the model. If the data are too poor for one model, they are most certainly too poor for any other model.

No one model can be used to solve the great variety of planning and design questions "better" than the other models. Literature discussions of model comparisons are useful to someone who needs to become aware of the types of models that exist and the types of problems that can be addressed by the great variety of models available. Loague and Freeze (1985) argued that an experienced modeler familiar with the specific study objectives is much more valuable than an untrained person relying on "objective analyses of modeling methods" such as presented in their paper. A comprehensive review of urban runoff models is well beyond the scope of this dissertation research, but the several methods commonly used to estimate urban runoff flows from impervious surfaces during small storms and the associated washoff of particulates are discussed in the text and compared to the methods proposed resulting from this research.

The required use of a model should be the most important factor in selecting a model. Linsley (1982) stressed that any model selected should be capable of providing the answers required.

A model should not be selected or rejected based on its given descriptive adjectives (empirical,

conceptual, black box, continuous, etc., as earlier described). However, if the model does not have sufficient documentation to determine its applicability to the objective being considered, then it should be rejected until sufficient information can be obtained. If the only models available for answering the specific study objectives are too expensive to run or to obtain the needed input information, then possibly no model should be used. "Bad answers are worse than no answers at all", "garbage in, garbage out", etc., even though trite, may still be applicable guidelines.

During model selection, it is important not to select a complex model simply because it considers many processes or requires substantial input data. Loague and Freeze (1985) found, in their comparison of several types of hydrology models, that less data intensive models based on simple regression relationships provided predictions as good as, or better than, more physically based models.

It may be best to take a gamble and develop a new model if existing models do not seem to be appropriate. Needless to say, model development is likely to require much more time and effort than originally thought. The main objectives of any model development effort should be to obtain the needed answers more efficiently than if using available models. A model should not be developed solely to be more comprehensive (and therefore possibly more complex) than available models, but to do the job better.

An example of the process in developing a new model is illustrated by the history of the Source Loading and Management Model (SLAMM)(Pitt 1986). It was conceived in the mid 1970s mostly as a data reduction tool for use in early street cleaning projects (Pitt 1979). Special field studies were designed and conducted in conjunction with many separate field projects to obtain necessary information (mostly source area sheetflow characterization and control measure performance information). After substantial work with the initial versions of the "model", it was decided that a needed management tool (at the decision making or planning level) could be developed. SLAMM was therefore expanded to enable other management practices (located at source areas and at the outfall) to be evaluated. A preliminary description of SLAMM was included in the Castro Valley NURP report (Pitt and Shawley 1982), but it was not until the other NURP projects and the field studies conducted as part of

this dissertation research were completed that a more comprehensive version of the model was able to be finished and tested. The whole process took more than ten years and the model directly used the benefits from more than several million dollars worth of field research.

The basis of SLAMM is to continually develop mass balances for both particulate and dissolved pollutants and runoff volumes for different proposed development and rain characteristics. It was designed to give relatively simple answers (pollutant mass discharges and control measure costs for a very large variety of potential conditions). It is therefore used as a planning tool, such as to generate the information needed to make planning level decisions, while not generating superfluous information unnecessary for these planning decisions.

Many alternative stormwater management models are available that can generate predicted outfall conditions with great resolution, as an example, but this information is of little value to planners and substantially increases the data gathering and computational costs.

G. Contributing Source Areas

Simple relationships used in estimating runoff volumes (and pollutants) that have many characteristics similar to the hypothesized urban runoff model presented in this dissertation include the concept of contributing source areas. Two major concepts of contributing source areas that have been discussed in the literature, partial areas and variable areas, are summarized here. Partial contributing areas assume that a relatively constant (but usually small) portion of a watershed is responsible for the flows reaching the receiving waters for all rains. Variable contributing areas, on the other hand, also typically assume a small flow contributing area, but the contributing area size varies for different rains. As shown in the results discussion of this dissertation, either of these concepts could be assumed by examining typical urban runoff field data, depending on the land cover make-up of the watershed. In concept, the hypothesized model resembles variable contributing area hydrology, but can result in runoff contributing area estimates that resemble partial contributing area hydrology.

Viessman et al. (1970) recommended that separate considerations be made of impervious and pervious areas in runoff analyses of small urban areas (in contrast to using a simple "lumped" model such as the rational model for the complete combined urban area). Many simple urban runoff models go to the extreme in separate analyses by assuming a partial contributing area process, with only the directly connected impervious areas contributing flows. This approach generally ignores losses from these impervious areas after the initial losses are satisfied and assumes that runoff very rarely occurs from pervious areas. The lack of importance of pervious areas is required to account for the long term losses that actually occur from the impervious areas.

Much hydrology research has been directed to the concept of contributing source areas. Section 3 discusses the importance of knowing the sources of runoff flows and pollutants before an effective runoff control program can be designed. Feyder (1979) concluded that "the seemingly overwhelming job of controlling nonpoint pollution is greatly simplified if the pollution sources are limited to distinct portions of the watershed". Hawkins (1982) gave several examples where only relatively small portions of range watersheds were contributing flows (and thus erosion) for different rain intensities. Grimmond and Oke (1986) showed that careless urban area irrigation was responsible for much of the water sources observed during dry weather.

Partial area overland hydrology studies were summarized by Freeze (1974). An overall conclusion from many studies noted that for vegetative rural watersheds in humid areas, water reaching the streams during the storms were only originating from a small (usually only 1 to 3 percent) and relatively constant portion of the watersheds. He also summarized variable area hydrology studies which concluded that the flow contributing source areas were also small, but changed in area depending on the rain characteristics. Hortonian overland flow mechanisms (runoff when rain intensity exceeds soil infiltration rate, up to the soil saturation limit) were thought to be most important in the "constant" partial areas. Subsurface flows were thought to be the most important mechanism in feeding the extended channels assumed in variable source areas. Therefore, the distribution of soil types is very

important in partial area hydrology (relying on Horton runoff).

In urban areas, the variable source area concept was found to be of significance in this research. However, the importance of the distribution of soil types was replaced by the distribution of land cover types (mostly pervious versus impervious areas). The source areas were found to vary, but for many areas significant variations were only identified by the hypothesized model when the rain volumes exceeded about 50 mm. Contributing surface areas of land uses having large amounts of impervious surfaces (such as shopping centers) changed very little, irrespective of the rain conditions. Because of the highly variable nature of urban areas, the portion of the complete area contributing flows and pollutants was therefore found to vary significantly. For shopping centers, almost all of the watershed areas were estimated to contribute flows and pollutants for all rains, while in low density residential areas flows were only contributed from the streets and driveways for almost all rains. For many pollutants and many land uses, the pervious areas were very important and could not be ignored as significant flow or pollutant sources for many rains.

Arnell (1982) suggested a simple method to estimate the fraction of watershed contributing flows in urban areas. He plotted rainfall versus runoff volumes (in a similar manner as described in the hypothesized model examined in this dissertation) and determined regression relationships. The slope of the regression line was estimated to be the portion of the area contributing runoff. Arnell assumed that if long term losses from urban contributing source areas were insignificant, then all of the rain falling on that area was reaching the receiving water. This dissertation research found that significant long term losses were likely to occur from all contributing areas, even "impervious" areas, the areas contributing runoff are also probably experiencing significant runoff losses. Therefore the area contributing runoff determined from the regression line slope method suggested by Arnell is actually much smaller than the actual area contributing runoff for most land uses. The important difference is the long term losses (mostly infiltration) that does occur from paved surfaces after initial losses are satisfied. In older areas that have been repaved many times (and are in good condition, without pavement cracks) and for roofs, the major

losses were found to be initial (before runoff begins) and the long term losses after runoff starts were found to be insignificant. However, most paved surfaces experience significant runoff losses after the initial losses are satisfied and runoff is affected by both initial and long-term losses (including infiltration losses, either through the pavement itself, or through pavement cracks). Urban areas are further complicated because of the presence of infiltration losses affecting runoff from distant (unconnected) impervious areas as the runoff from impervious areas flow across pervious areas before reaching the drainage system.

Miller (1984) suggested that a breakpoint rainfall volume existed where different urban surfaces contributed flows. This breakpoint was also identified through a similar regression analysis as described above between basin rainfall and runoff. Miller described this breakpoint as occurring when the land area surface flow system becomes saturated. It is identified by fitting two first order polynomials (straight lines) to the data for areas having identified flow channels, or a straight line and a curved line when sheetflow predominates. The breakpoint occurs where the two lines cross. The hypothesized model presented in this dissertation uses a single second order polynomial (curved line) to fit the data. Because of the typical scatter in the data, identifying the crossing location seems to be very sensitive to errors.

Draper and Smith (1981) described how two intersecting lines could be fitted to the data, using trial intersection points; but it would be very difficult to identify the "best" curve fit or to show that two straight lines fit the data significantly better than a single curved line. Because of the typical data scatter, regression analysis usually only identifies the intercept (indicating initial losses) and the slope functions of first order models as being significant. Second order models are only significant for areas having large amounts of pervious areas and when the rains observed cover a wide range of depths . It would be very difficult to show that any specific intersection point of two first order polynomials is significantly better than any other intersection location.

Even with these regression analysis problems, the breakpoints identified by Miller (1984) varied from about 25 mm for a residential test basin to about 50 mm

for apartment and highway test basins. These values are in the general range of rains found to produce significant runoff from pervious areas during this dissertation research. However, the breakpoint rain values produced in the hypothesized model are smaller for areas having small amounts of pervious areas located close to the drainage system (such as for most residential areas) and larger for areas having large pervious areas located farther from the drainage system.

H. "Non-Linear" Processes

Straight line relationships between rainfall and runoff are sometimes confusingly termed linear relationships. Similarly, non-linear relationships refer to models that do not contain straight lines, instead of the correct regression meaning referring to the placement of the equation coefficients.

In many cases, especially for highly impervious urban areas, such as shopping centers, and for limited ranges of rain observations, straight lines relating runoff to rain are the obvious "best" relationship.

Rainfall-runoff observations of largely pervious areas are also usually described with straight lines because of the rarity of large rain and runoff observations that are greater than the "breakpoint" rain volume, typically 25 to 50 mm of rain.

This dissertation research examined two widely different test areas for a large number of rains having a wide range in rain quantities to confirm the hypothesized "curved" relationship between rainfall and runoff. Data from other field studies that contained large events (approaching 100 mm of rain) were also examined in this research. Even though this research was directed towards small storm hydrology and washoff processes, these rare large rain observations were needed to confirm the hypothesized model behavior during extreme conditions (when the long term runoff losses were finally satisfied).

I. Significant Rainfall-Runoff Factors

Many urban runoff studies have used step-wise regression analysis to identify important variables affecting runoff from urban areas. As an example, a

significant effort is currently being undertaken by the USGS in examining the NURP runoff data (Driver and Lystrom 1986). This effort is hampered (like many similar efforts) by a lack of information concerning potentially significant variables. These types of step-wise regression analyses are usually more informative when conducted as part of the local initial research effort. The USGS large scale effort may indicate significant regional variations, but the lack of an in-depth knowledge of the test basins and the large inherent errors in measuring rainfall and runoff in urban areas may mask or even alter these regional conclusions.

A common error in regression analysis is the inclusion of several closely related independent variables in the final "model". One example of this is using average rain intensity at the same time as total rain volume and rain duration as independent parameters in the model. In many cases, either intensity or volume and duration should be included; all three parameters should not be included. Strange things may happen if intensity is included with either one of the other two parameters. Spurious self-correlations may also occur between the dependent and independent variables of the model. One way this may occur is when parameters having widely different numeric magnitudes are combined as a single parameter and are regressed against the larger of the two parameters. Examining model residuals resulting from any regression analysis (linear and non-linear) is also not usually completed to determine if any other regression assumptions are violated (Draper and Smith 1981). Unfamiliarity or ignoring these regression assumptions are the cause of many modeling errors. The automatic use of step-wise regression analysis is probably of most concern, especially when not enough thought has been given to separating dependent and independent variables (and their combinations). The following paragraphs summarize selected studies that have used regression analysis to identify the important variables affecting urban runoff hydrology.

Rallison and Miller (1982) summarized early studies that proposed the significant variables affecting runoff. Sherman (1949) was one of the first to plot direct runoff versus storm rainfall, while Mockus (1949) suggested a list of important parameters that is surprisingly comprehensive: soils (types, areal extents, and locations), land use (kinds, areal

extents, and locations), antecedent rainfall, duration of a storm and the total rain, and annual average temperature and date of the storm.

Many recent regression studies have identified significant independent variables that are conveniently divided into site or rain characteristics. Site characteristics may include percent perviousness, percent directly connected imperviousness, the type of drainage system (grass swales versus concrete curbs and gutters), roof connections, areas of specific land covers and their distances from the drainage system, hydrologic soil types (specifically their infiltration and moisture capacities), depth to groundwater, etc. Rain characteristics may include total rain, average rain intensity, different measures of peak rain intensity, rain duration, time since last rain, antecedent rain conditions, etc. The dependent runoff variables of concern may include total runoff volume (typically the most important variable for water quality studies), and different measures of runoff rate (most important for drainage design studies), time lag between start of rain and runoff, and runoff duration. The list of potential independent variables can become quite long; step-wise regression is a useful tool to identify the important variables. As stated previously, care must be taken to identify the inter-relating independent variables and to select the appropriate transformations to attempt in the regression. In all cases, the resulting regression model residuals must be examined to confirm the suitability of the regression assumptions.

Few studies have investigated such a wide range of variables as listed above. Even fewer studies have investigated a diverse range of variables over a wide geographical area (such as Driver and Lystrom 1986). However, a few notable conclusions are apparent from the more comprehensive studies based on predicting total runoff volume. Rain volume is the most important variable when predicting runoff volume for a specific site, while the percent imperviousness is the most important site characteristic when predicting runoff volumes for different sites for the same rain (Baun 1979 for the Milwaukee IJC data, Driver and Lystrom 1986 for the complete NURP data set). Not many studies examined the presence of grass swales as a variable, but Baun (1979) found them to be the most important site variable. Unfortunately, Driver and Lystrom (1986) did not have that information available for all of the

NURP sites. The NURP data (EPA 1983) was confused because of inconsistent site characterizations concerning the meaning of "directly connected impervious" areas at study sites that were drained by swales.

Many studies have concluded that rain intensity should be a significant variable needed when predicting total runoff volume (assuming Hortonian runoff). Very few regression studies using significant field data have found this to be true (such as this research and Pratt and Henderson 1981). Rain intensity does not vary widely for most areas, and the limited data sets typically are restricted to a narrow range of observed rain intensity values. During this dissertation research in Toronto, as an example, it was found that most rains have average intensities between 1 and 10 mm per hour, with peak 5-minute intensities seldom exceeding 25 mm per hour.

Most of the urban areas that have been studied are also characterized by large expanses of impervious areas or heavily disturbed soils. The hydrologic soil characteristics obtained from soil maps (based on native undisturbed soil conditions) typically used in regression studies therefore have little meaning in such environments. Actual soil infiltration rates need to be measured in the study areas before this seemingly important soil characteristic can be suitably evaluated. However, as found during this study, rain intensity is very important in predicting runoff flow rates.

Very good predictions of total runoff volume are possible by knowing only total rainfall, after a regression model has been developed for a specific site. Baun's analysis (1979) of the Milwaukee area IJC data is an example of a relatively complete study where 28 to 32 rain events were examined at eight different sites, half drained by grass swales (including an airport, a shopping mall, a freeway interchange, and five residential and commercial areas). Because of the multi-year nature of the data collection effort, the range of rain conditions included in this analysis was also impressive (0.5 to 95 mm). Still, Baun could not statistically justify including second-order, or higher, polynomial expressions for total rain in his prediction equations.

Almost all of the rains included in any monitoring effort will be less than 25 mm, with the larger events (approaching 100 mm of rain) being very rare. The data

assumptions for regression analysis require that the independent variables (rain depth) be normally distributed throughout their range. Because of the difficulty of obtaining large rain event data, and because of the lack of negative rain values, most of the observations are clustered close to the small observations. The few large event values therefore do not influence the regression results very much. Transformations of the variable values can be used to help satisfy the normal assumption, if done carefully. The EPA (1983) found that log transformations of the NURP data resulted in distributions that approached a normal distribution over much of the data range (typically from about 5 to 95 percent on a normal plot). Unfortunately, few modelers have attempted to use adequately transformed data in their regression analysis.

Adequately transformed data would allow the larger rain observations to exert a justifiable leverage effect on the abundant smaller event data. The proper influence of the rare large events would result in more curvature (as reflected by significant second-order polynomial coefficients) in the relationship between rain and runoff. This curvature is generally more common for models that are log-transformed, include large rain events, and for study areas having small amounts of impervious areas. Because the runoff volume is "close" to the rainfall depth for areas that are mostly impervious due to generally small runoff losses, very little curvature in the relationship is expected. For areas having much greater runoff losses, the errors associated with measuring the runoff losses become small in relation to the loss values, and the curvature terms are more easily shown to be significant. This dissertation research explored the conceptual meaning of this curvature in detail.

Because of the small relative sizes of the runoff losses in relationship to the rain depth (for moderate and large rains), a constant runoff coefficient is typically used for impervious areas. For many purposes (such as conservative drainage design that is mostly concerned with "large" rains) this simplified approach is justified. However, for water quality analyses where more emphasis must be placed on the small events, this approach has been responsible for many incorrect conclusions regarding the importance of the relative contributions from different source areas and the effectiveness of source area controls.

Shelley and Gabary (1986) introduced a model they are developing for the Federal Highway Administration that makes extensive use of a constant runoff coefficient for impervious surfaces. Plots of the considerable highway runoff data available showed that Rv (the volumetric runoff coefficient, or runoff volume/rainfall volume) plotted against rainfall indicated a reasonably constant Rv value for sampling locations that were mostly paved. The exceptions, of course, were the small rains that had runoff volumes grossly over-estimated when using a constant Rv.

Similar Rv versus rainfall plots by Sautier (1983) summarized many Swiss urban area runoff observations. He recommended using much smaller constant Rv values than typically used in the US for most "impervious" surfaces. His Rv recommendations for concrete and asphalt streets were 0.8 (compared to typical Rv values of 0.9 to 0.95 used in the US). If the street does not have waterproof joints, the Rv value decreases dramatically to 0.5, reflecting significant infiltration through pavement seams. An examination of his plots for rooftops showed that asphalt, asbestos-cement, and tile roofs had Rv values of about 0.8 to 0.95, but only after several millimeters of rain.

Interestingly, his Rv values for flat gravel roofs was only 0.25, reflecting considerable losses. Swiss gravel roofs must contain much deeper gravel layers than in the US.

An evaluation of the possible errors associated with using constant Rv values was conducted as part of this dissertation research, using the Milwaukee NURP runoff data presented by Bannerman et al. (1983). More than 400 rain events were examined at four sets of paired land use sites. Simultaneous plots of runoff versus rainfall did not show any significant seasonal differences, but about 90 percent of the data was obtained during the spring and summer. The paired data sets were then compared with no significant differences noted between the paired monitoring stations. However, the four land use categories were found to have significantly different runoff responses. Therefore, about 100 data sets were available for each of four major land uses: medium density residential, high density residential, strip commercial (with some high density residential), and commercial parking lots (shopping centers).

The initial runoff analyses reported by Bannerman et al. (1983) were restricted to developing constant

(mean) Rv values, but these subsequent regression analyses indicated substantial differences in runoff predictions between the two methods. Table C.1 is a summary of the possible errors if a constant Rv value is used instead of the regression equations. This table shows that very large over-prediction errors (at least 50 to more than 500 percent) of runoff volume would occur for the smallest rains at all sites, but especially at the more impervious sites. Similarly, important under-predictions of runoff volumes would occur for the larger events. Events accounting for more than 80 percent of the annual runoff volume would be in error by more than 10 percent at the medium density residential site. For all of the sites, events accounting for 14 to 35 percent of the actual total annual runoff would be in error by more than 25 percent if the constant mean Rv values were used instead of the regression equations.

Conflicting recommendations concerning runoff processes from impervious surfaces are still common after more than 20 years of research in urban area hydrology. Viessman in 1966 recognized the importance of source area runoff processes above the inlets before the outlet hydrology conditions could be quantified.

Table C.1 Errors Introduced by using Constant Rv Values

			Medium Density Residential (without alleys)				Medium Density Residential (with alleys)				Commercial/Medium Density Residential (with alleys)				Commercial Parking Lot			
(A)	(B)	(C)	(D)	(E)	(F)	(G)	(D)	(E)	(F)	(G)	(D)	(E)	(F)	(G)	(D)	(E)	(F)	(G)
0.05	4.9	0.4	<0.01(4)	<0.2	<0.20	>40	0.01	0.2	0.20	90	0.004	0.05	0.10	520	<0.01(4)	<0.1	<0.20	>260
0.1	18.3	3.2	0.02	1.8	0.17	65	0.03	2.3	0.30	27	0.04	1.9	0.40	55	0.04	1.6	0.38	89
0.3	26.0	13.8	0.09	11.7	0.29	-3	0.11	11.8	0.37	3	0.19	12.6	0.63	-2	0.23	13.2	0.70	-8
0.5	16.3	14.4	0.16	13.0	0.32	-13	0.20	13.5	0.40	-5	0.35	14.6	0.70	-11	0.43	15.4	0.86	-16
0.7	16.7	20.6	0.23	19.1	0.33	-15	0.30	20.7	0.43	-12	0.53	22.2	0.74	-16	0.62	22.0	0.89	-19
1.0	11.0	19.4	0.35	19.2	0.35	-20	0.46	29.9	0.46	-17	0.70	21.9	0.78	-21	0.92	22.3	0.92	-22
1.5	4.1	10.9	0.57	11.7	0.38	-26	0.79	13.4	0.53	-28	1.23	12.9	0.82	-24	1.40	12.6	0.94	-23
2.5	1.2	5.3	1.07	6.4	0.43	-35	1.64	8.1	0.66	-42	2.27	6.9	0.91	-32	2.38	6.3	0.95	-24
3.5	0.8	4.9	1.64	6.5	0.47	-40	2.73	9.0	0.78	-51	3.45	7.0	0.99	-37	3.35	5.9	0.96	-25
5.0	0.8	7.1	2.67	10.7	0.53	-47	—(3)	—	—	—	—	—	—	—	—	—	—	—
* —	100.0	100.0	—	100.1	0.28(1)	—	—	99.8	0.38	—	—	100.0	0.62	—	—	100.1	0.72	—

(A) Rain Increment (inches)
(B) Percentage of Annual Rain in Category
(C) Percentage of Annual Rain Volume in Category
(D) Runoff (inches)(3)
(E) Percentage of Annual Runoff Volume in Category
(F) Rv
(G) Percent Error in Runoff Calculation (2)

*Annual Means

(1) Weighted annual mean Rv based on % of rains in each category.
(2) Error in runoff volume by using constant mean annual Rv value: $\text{error} = \frac{\text{Rv mean} - \text{Rv variable}}{\text{Rv variable}}$

(3) Beyond limit of regression equations
(4) Negative runoff volumes are not "allowable," so negative predicted flows using the regression equations are considered as <0.01 inches of runoff.

Brater in 1968 identified impervious area hydrology as being of the utmost importance when he introduced the concept of "hydrologically significant impermeable areas". As stated elsewhere, most of the early urban hydrology investigations were concerned with flooding and drainage design, and therefore stressed large rain events. Unfortunately, many of the simplifications appropriately applicable to smaller rains for flooding analysis have been incorrectly applied to urban runoff quality studies that must also examine small, common events in detail.

J. Rational Method

The rational method is the most popular procedure used to estimate peak flow rates. It is easy to use and may be sufficiently accurate for some purposes. Linsley (1982) credited Perrault in 1674 with the concept of estimating streamflow (the Seine River near Paris) as a percentage of precipitation. Roberts was using runoff coefficients in 1844 in Ireland for drainage design, and Mulvaney described the rational formula in a paper given to the Civil Engineers of Ireland in 1851

(Linsley 1982). The rational method was introduced in the US by Kuichling in 1889 (Trans. ASCE). The rational method relates peak runoff rates (Q) to the critical period average rainfall intensity (I):

$$Q = C\ I\ A$$

where C= runoff coefficient
A= drainage basin area

The critical rain intensity averaging period is the time necessary for water to flow from the most distant location of the drainage system to the point of interest (the time of concentration). The selection of the C value is difficult because it depends on many conditions. Table C.2 summarizes some commonly used values. These values are usually used without regard to rain intensity, even though they are most applicable for 5 to 10 year storms. They are not very accurate for common, small storms that are of interest in many urban runoff quality studies.

Hromadka (1982) listed three basic assumptions that must be satisfied to properly use the rational formula:

Table C.2 Range of Coefficients, Classified with Respect to the General Character of the Tributary Area

Description of Area	Runoff Coefficients
Business	
Downtown	0.70 to 0.95
Neighborhood	0.50 to 0.70
Residential	
Single-family	0.30 to 0.50
Multi-units, detached	0.40 to 0.60
Multi-units, attached	0.60 to 0.75
Residential (suburban)	0.25 to 0.40
Apartment	0.50 to 0.70
Industrial	
Light	0.50 to 0.80
Heavy	0.60 to 0.90
Parks, cemetaries	0.10 to 0.25
Playgrounds	0.20 to 0.35
Railroad yard	0.20 to 0.35
Unimproved	0.10 to 0.30

It often is desirable to develop a composite runoff coefficient based on the percentage of different types of surface in the drainage area. This procedure often is applied to typical "sample" blocks as a guide to selected of reasonable values of the coefficient for an entire area. Coefficients with respect to surface type currently in use are:

Character of Surface	Runoff Coefficients
Pavement	
Asphaltic and Concrete	0.70 to 0.95
Brick	0.70 to 0.85
Roofs	0.75 to 0.95
Lawns, sandy soil	
Flat, 2 percent	0.05 to 0.10
Average, 2 to 7 percent	0.10 to 0.15
Steep, 7 percent	0.15 to 0.20
Lawns, heavy soil	
Flat, 2 percent	0.13 to 0.17
Average, 2 to 7 percent	0.18 to 0.22
Steep, 7 percent	0.25 to 0.35

The coefficients in these two tabulations are applicable for storms of 5- to 10-year frequencies. Less frequent, higher intensity storms will require the use of higher coefficients because infiltration and other losses have a proportionally small effect on runoff. The coefficients are based on the assumption that the design storm does not occur when the ground surface is frozen.

Source: Lazaro 1979

"1) the frequency of the storm runoff is the same as the return frequency of rainfall producing the runoff,
2) the peak runoff rate occurs when all parts of the drainage area are contributing to the runoff, and
3) the design rainfall is uniform over the watershed area tributary to the point of concentration, and the intensity is essentially constant during the storm duration equal to the time of concentration."

Rossmiller (1982) stated that various attempts to simplify the selection of the coefficient contained in the rational formula has resulted in some common misconceptions and irregular coefficient selections for different users. One of the most common problems has been the use of the rational formula to estimate runoff volume. Others have incorrectly used the rational formula to derive hydrographs. The rational formula is limited to estimating only a single point of the hydrograph, the peak.

The major difficulty in using the rational formula is selecting an appropriate coefficient value. Schaake et al. (1967) and later Johnson and Meadows (1980) reported that the coefficient needs to be varied for different design storms for the same site. Land use descriptions alone are not sufficient in selecting an appropriate coefficient. Rossmiller (1982) concluded that the coefficient should be increased for larger

(less common) storms. However, Sautier (1983) found that published coefficient values are generally too high for urban areas.

Morris (1983) compared several simple urban hydrology models using runoff and rain data from 25 USGS urban monitoring sites in Ohio. He found that the rational formula generally over-estimated peak flow, but performed best for small drainage areas (less than 100 acres) and for small rains (less than 1 inch). He concluded that the rational formula could be considered adequate for small areas and for rains having recurrence intervals of about 2 to 5 years. Therefore, it must be used with care in designing urban drainage systems, which are usually based on 10 to 25 year recurrence interval storms, especially for large drainage areas.

An early paper by Ardis et al. (1969) surveyed the drainage design procedures used by many cities. By far, the most common method used was the rational formula. In the 20 years since this report, it appears that the rational formula is still commonly used for drainage system design. Because of its popularity, Ardis et al. devoted much of their paper to a discussion of the ways the rational formula was being misused. The largest problems were in misusing rain intensity (not averaged for the correct time of concentration) and coefficient values, and in combining subarea runoff rate results.

K. Soil Conservation Service Curve Number Procedure

The SCS generalized runoff relationship is based on a plot of rainfall and runoff versus time. Runoff (Q) begins after an initial abstraction (Ia) is satisfied and then is equal to rainfall (P), minus infiltration (F) for separate time periods of runoff (SCS undated). At any time, therefore;

$$Q = P - Ia - F$$

S is defined as the maximum potential total abstraction (including both initial losses and infiltration). S is limited either by the rate of infiltration at the ground surface or the amount of water storage available (Rallison and Miller 1982). The following relationship is assumed for the simplest case where runoff begins immediately with rainfall (no initial abstractions):

$$F/S = Q/P_e$$

where P_e is the potential runoff (storm rainfall minus initial abstraction, ignoring infiltration). Therefore:

$$F = P_e - Q, \text{ and rearranging;}$$

$$Q = P_e^2 / (P_e + S)$$

The SCS (undated) has estimated that Ia = 0.2S, based on monitoring data from small watersheds. Therefore:

$$P_e = P - Ia = P - 0.2S, \text{ substituting leaves;}$$

$$Q = (P - 0.2S)^2 / (P + 0.8S)$$

The curve number (CN) is derived from S and is defined as:

$$CN = 1{,}000 / (S + 10)$$

Runoff volume is therefore directly related to rainfall volume and the curve number. Table C.3 shows the runoff

Table C.3 Runoff Depths (inches) for Selected Curve Numbers and Rainfall Depths

Rainfall (inches)	Curve Number (CN)[1]								
	60	65	70	75	80	85	90	95	98
1.0	0	0	0	0.03	0.08	0.17	0.32	.56	.79
1.2	0	0	0.03	0.07	0.15	0.28	0.46	.74	.99
1.4	0	0.02	0.06	0.13	0.24	0.39	0.61	.92	1.18
1.6	0.01	0.05	0.11	0.20	0.34	0.52	0.76	1.11	1.38
1.8	0.03	0.09	0.17	0.29	0.44	0.65	0.93	1.29	1.58
2.0	0.06	0.14	0.24	0.38	0.56	0.80	1.09	1.48	1.77
2.5		0 30	0.46	0.65	0.89	1.18	1.53	1.96	2.27
3.0	0.33	0 51	0.72	0.96	1.25	1.59	1.98	2.45	2.78
4.0	0.76	1.03	1.33	1.67	2.04	2.46	2.92	3.43	3.77
5.0	1.30	1.65	2.04	2.45	2.89	3.37	3.88	4.42	4.76
6.0	1.92	2.35	2.80	3.28	3.78	4.31	4.85	5.41	5.76
7.0	2.60	3.10	3.62	4.15	4.69	5.26	5.82	6.41	6.76
8.0	3.33	3.90	4.47	5.04	5.62	6.22	6.81	7.40	7.76
9.0	4.10	4.72	5 34	5.95	6.57	7.19	7.79	8.40	8.76
10.0	4.90	5.57	6.23	6.88	7.52	8.16	8.78	9.40	9.76
11.0	5.72	6.44	7.13	7.82	8.48	9.14	9.77	10.39	10.76
12.0	6.56	7.32	8.05	8.76	9.45	10.12	10.76	11.39	11.76

(1) To obtain runoff depths for CN's and other rainfall amounts not shown in this table, use an arithmetic interpolation.

Source: SCS 1986

volumes expected for different rainfalls and curve numbers (SCS undated).

Hjelmfelt et al. (1982) and Bales and Betson (1982) derived an equation where an "observed curve number" could be determined from field observations of rainfall and runoff. Assuming Ia = 0.2S, they determined:

$$S = 5P + 10Q - 10(Q^2 + 1.25QP)^{1/2}, \text{ leading to:}$$

$$\text{Observed CN} = 1000/[10 + 5P + 10Q - 10(Q^2 + 1.25QP)^{1/2}]$$

Hjelmfelt et al. (1982) stated that observed CN values for actual rainfall and runoff values leads to a wide range in CN values for a single test site. This dissertation research examined the initial losses and the long term variable losses in detail and found that the initial losses (Ia) were not related to S for impervious areas.

The Source Loading and Management Model (SLAMM) calculates the observed CN values for each modeled event using this procedure as an interface between SLAMM and models that are commonly used to design drainage systems. SLAMM has also shown how many site development variables affect the CN, in addition to the criteria contained in the common simple lists of suggested CN values for gross development conditions.

Walesh (1982) stated that the SCS initial abstraction assumptions may be adequate for severe storms, but may not be adequate for the small, frequent storms of most interest in water quality studies. Hromadka et al. (1983) also questioned the SCS initial abstraction term and suggested that the Ia value is actually about 1/4 to 1/2 of the value used by the SCS. For large design storms for urban drainage and flooding studies, they assumed that Ia was satisfied by previous rainfall and was therefore zero. Chen (1982) found that Ia may vary from zero to S, for the same drainage area, depending on rain intensity. Immediate ponding with instantaneous runoff would result in an Ia value of zero, while a situation resulting in no runoff would imply an Ia value equal to S. Chen assumed these conditions to be dependent on rainfall intensity, but this dissertation research found Ia to be mostly dependent on total rainfall depth and micro-scale detention storage volume. He concluded that it would be reasonable to ignore Ia for large rains in areas having CN values greater than 70. Aron (1982) also questioned

the assumption that Ia would be directly related to S and questioned further the assumption that Ia accounts for 20 percent of S. For a 3.33-inch rain with a dense residential development, he showed that the first 0.67 inches of rain would fall with no resulting runoff.

Several researchers have investigated the relationship between the SCS runoff model and other infiltration models. Hjelmfelt (1980) found that the SCS curve number runoff equation was identical to the Holton-Overton infiltration equation for the special case of a storm having constant rain intensity and a zero continuous infiltration rate. The SCS model assumes that infiltration approaches a zero value during long storms instead of an "expected" constant terminal infiltration rate. Chen (1982) found that the SCS model can be used as an alternative expression of the infiltration decay curve, except for extreme conditions. He found that as the CN decreases, Ia and rainfall changes increasingly affect runoff prediction. He stated that the ratio equation ($F/S = Q/P_e$) upon which the SCS curve number equation was derived was assumed and was still not validated. Aron (1982) found that the SCS method allows much easier selection of infiltration parameters than other infiltration models (such as the Horton equation). He did recommend that the Ia value be reduced to somewhere between 5 and 10 percent of the soil storage capacity and a minimum infiltration loss rate be used (instead of assuming zero as the steady-state infiltration value). Minimum loss rates suggested are 0.4 inches per hour for class A soils, 0.24 inches per hour for class B soils, 0.12 inches per hour for class C soils, and 0.08 inches per hour for class D soils. Obviously, these suggested changes in initial abstraction and final infiltration rates preclude using the tables and figures prepared by SCS that relate runoff to CN and rain. When the equations are used in computer models, however, these suggested changes can be easily accommodated.

CN selection is very important when using the SCS curve number method. Sabol and Ward (1983) found that the SCS CN method is not as sensitive to rainfall as it is to CN selection (on an equal percentage error basis). Bales and Betson (1982) found that observed CN values were not correlated to any of the physical basin characteristics that they examined. However, the observed CN values were fairly well linearly correlated with land use characteristics (such as percent urbanized and the degree of storm sewer use). Geologic

and soil characteristics were not as well correlated with the observed CN values as land use characteristics, but were better correlated than with the physical basin characteristics. Rawls et al. (1981) also found that runoff estimates were sensitive to land use classification, but were not very sensitive to the method used to average soils and land cover data for small to medium sized drainage areas. Rallison and Miller (1982) reported that CN values vary by storm duration (storm depth ?), becoming smaller as the storm duration (depth ?) increases.

Another problem with the SCS CN method reported by several authors relates to the antecedent moisture condition (AMC) (Hope and Schulze 1982). The AMC values change by discrete increments, based on the amount of rain in the preceding five days. When the CN equations are used for a series of events, sudden shifts in CN values (and in the estimated runoff) occur when the AMC moves from one category to another. They also are bothered by the arbitrary selection of five days as the accounting period when determining AMC.

Several authors have reported that the SCS CN method may severely underestimate the runoff volume for small events (Ontario 1984). Sabol and Ward (1983) examined the SCS CN method on undeveloped sites and on urbanized lawns in Albuquerque using rainfall infiltrometer experiments. Green-Ampt infiltration equation parameters were found to be reasonable, but the observed curve numbers were substantially different than expected. The observed rainfall-runoff data did not follow a constant CN line: the observed CN values decreased during the rainfall. Initial CN values of about 95 at the beginning of a rain decreased to CN values of about 78 after 4 inches of rain. CN values of about 73 decreased to CN values of about 58 when the rainfall increased from 1 inch to 4 inches.

Rallison and Miller (1982) stated that urban area CN values are based on interpretive values and not from actual monitoring data. They concluded that the CN procedure does not work well in areas of Karst topography or in any area where a large proportion of the runoff is subsurface. The CN method may so result in errors where only a portion of the watershed is contributing flow or when there is a significant variation in rain intensity over the drainage area.

Since the early 1970s, the SCS CN procedure has been increasingly used to investigate hydrology problems (especially in ungauged watersheds) that it

originally was not intended to solve (Rallison and Miller 1982). The SCS CN equations do not contain any expressions for time. It was developed to estimate total runoff from individual storms and not to produce hydrographs. Morel-Seytoux et al (1982) found that the CN method is still very useful, even if based on questionable assumptions. They stated that;

> "a wrong model with a wrong parameter can still provide decent results. Coded in the CN is a lot of good information (actual rainfall-runoff data)."

L. Unit Hydrograph

The unit hydrograph approach has been used to overcome some of the problems of the rational and SCS CN methods. However, it was conceived as a method to help in designing drainage systems, not to investigate urban runoff quality problems. It is a linear model, like the rational method, but it results in an estimate of the distribution of runoff flows throughout a rain event, instead of just the peak flow rate. Individual unit hydrographs, after adjusting for volume, can be combined along a time scale to result in a complex expected hydrograph for a typical storm.

Espey et al. (1977) credited Folse with originating the unit hydrograph concept in 1929. Espey et al. also stated that the Boston Society of Civil Engineers in 1930 appears to have defined its major concept thus; "the base of the flood hydrograph appears to be approximately constant for different floods, and peak flow tends to vary directly with the total volume of runoff". It was presented by Sherman in 1932 (Eng. News Rec.) as an idealized presentation of flow rates resulting from 25 mm (1 inch) of runoff. Hromadka (1982) defined the major concepts behind the unit hydrograph as; "the assumption that watershed discharge is related to the total volume of runoff, and that the time factors which affect the unit hydrograph shape are invariant".

Hormadka (1982) summarized the four basic assumptions associated with unit hydrograph development and use as follows:

> "1) the critical storm rainfall pattern is uniformly distributed throughout the watershed,
> 2) there exists a direct proportionality between watershed runoff and the effective rainfall volume,
> 3) for any volume of effective rainfall occurring within a specified duration, the resulting runoff hydrograph is of a constant duration, and
> 4) the basin unit hydrograph is invariant throughout the critical design storm."

These assumptions are not overly critical in relationship to other simplified urban hydrograph methods. The second assumption requires the use of a constant Rv value, with the attendant errors for small events described earlier. The development (selection) of the unit hydrograph is very dependent on these criteria, however.

Developing a unit hydrograph can be difficult as it requires monitoring a storm that had uniform rains over the entire watershed producing between 13 and 45 mm (0.5 and 1.75 inches) of runoff. Separate unit hydrographs must be developed for different time periods. Long time periods (up to 12 hours) are typically used for large drainage areas (about 1000 square miles), while 10-minute time periods are used for small urban drainages. Mays and Coles (1980) identified the major problem with developing a unit hydrograph as decomposing a multiperiod storm into component separate runoff hydrographs and then deriving the unit hydrograph. Very few individually monitored storms will satisfy the criteria listed above.

Unit hydrographs have been used in urban runoff studies to identify the significance of development on an existing (predevelopment) design storm hydrograph.

Willeke (1966) also stressed the need to modify the shape of the standard unit hydrograph for an area as the area's watersheds undergo urbanization.

Espey et al. (1977) has developed a useful set of equations, based on updating his earlier work (Tracor 1973), that describe the shape of urban area unit hydrographs. Hormadka (1982) also summarized Espey's work. Table C.4 presents these equations for the general 10-minute urban hydrograph shown in Figure C.1. The five most important watershed factors that determine the shape were found to be watershed area, distance from the study location to the most upstream boundary of the drainage area, the main channel slope, the imperviousness of the drainage area, and the conveyance efficiency (which only affects hydrograph rise time). The conveyance efficiency varies from 0.6 (extensive channel system with storm sewers and no channel vegetation) to 1.3 (natural channel conditions with heavy vegetation). An extensive channel system with an enclosed storm drainage system would have a conveyance efficiency of about 0.6, while a system with some channelization and with an enclosed sewer system would have a conveyance efficiency factor of about 0.8.

Table C.4 Ten-Minute Unit Hydrograph Equations

Equations	Total Explained Variation
$T_R = 3.1\ L^{0.23} C^{-0.25} I^{-0.18} O^{1.57}$	0.802
$Q = 31.62 \times 10^3 A^{0.96} T^{-1.07}$	0.936
$T_B = 125.89 \times 10^3\ A\ Q^{-0.95}$	0.844
$H_{50} = 16.22 \times 10^3\ A^{0.93}\ Q^{-0.92}$	0.943
$H_{75} = 3.24 \times 10^3\ A^{0.79}\ Q^{-0.78}$	0.834

L = is the total distance (in feet) along the main channel from the point being considered to the upstream watershed boundary.

S = is the main channel slope (in feet per foot) as defined by H/(0.8L), where L is the main channel length as described above and H is the difference in elevation between two points. A and B. A is a point on the channel bottom at a distance of 0.2L downstream from the upstream watershed boundary. B is a point on the channel bottom at the downstream point being considered

I = is the impervious area within the watershed (in percent)

O = is the dimensionless watershed conveyance factor.

A = is the watershed drainage area (in square miles).

T_R = is the time of rise of the unit hydrograph (in minutes).

Q = is the peak flow of the unit hydrograph (in cfs).

T_B = is the time base of the unit hydrograph (in minutes).

H_{50} = is the width of the hydrograph at 50% of the Q (in minutes).

H_{75} = is the width of the unit hydrograph at 75% of Q (in minutes)

Source: Espey et al. 1977

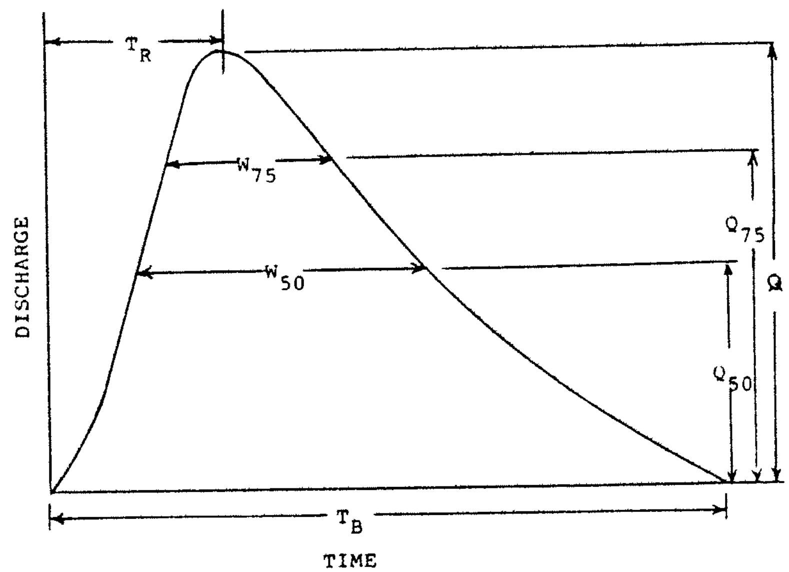


Figure C.1 Definition of Unit Hydrograph Parameters

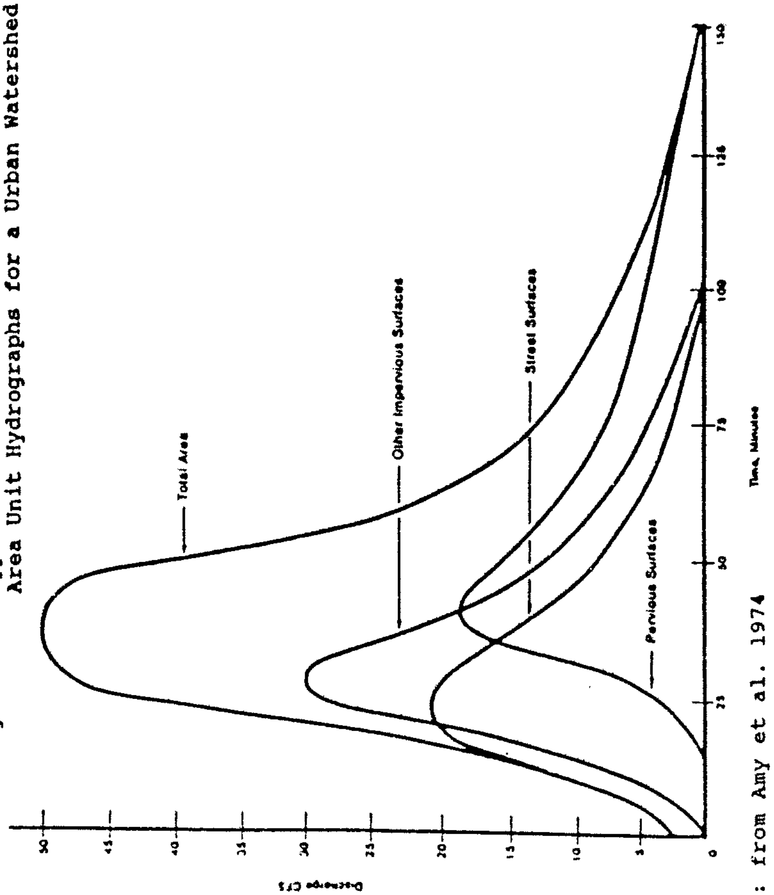
Source: Espey et al. 1977

A fundamental problem with unit hydrographs was described by Willeke (1966) and by Mays and Coles (1980) as the inability of a linear model to approximate the processes in a nonlinear system. This inability has been demonstrated by recognizing that different size storms have different unit hydrographs. Therefore, unit hydrographs should be used for only a narrow range of predicted conditions near the critical design storm. They therefore have limited use for wate quality studies where a wide variety of "design" storm (typically needed to estimate seasonal mass discharges must be evaluated. Knowledge of the variations in discharge conditions during a single event also has limited value in most water quality evaluations.

Amorocho (1961) stated that it was a well known fact that unit hydrographs derived from large floods usually differ from those derived from minor floods. H also suspected the inapplicability of the principle of superposition of unit hydrographs. Mays and Coles (1980) along with Hossan et al. (1978) also stated tha unit hydrographs vary somewhat for different rains fro the same drainage area.

Figure C.2 shows how urban source area unit hydrographs are combined to produce a complete

Figure C.2 Hypothetical Combination of Individual Source Area Unit Hydrographs for a Urban Watershed



Source: from Amy et al. 1974

hydrograph for the complete drainage (Amy et al. 1974 Directly connected impervious areas contribute the first flows, and more distant impervious areas and pervious areas contribute flows at a later time. Depending on the magnitude of the rain, some of these later components may never contribute to the total flow. Therefore, the overall shape of the outfall uni hydrograph is very dependent on the size of the storm which determines the contributing components.

M. Hydrodynamic Physical Models

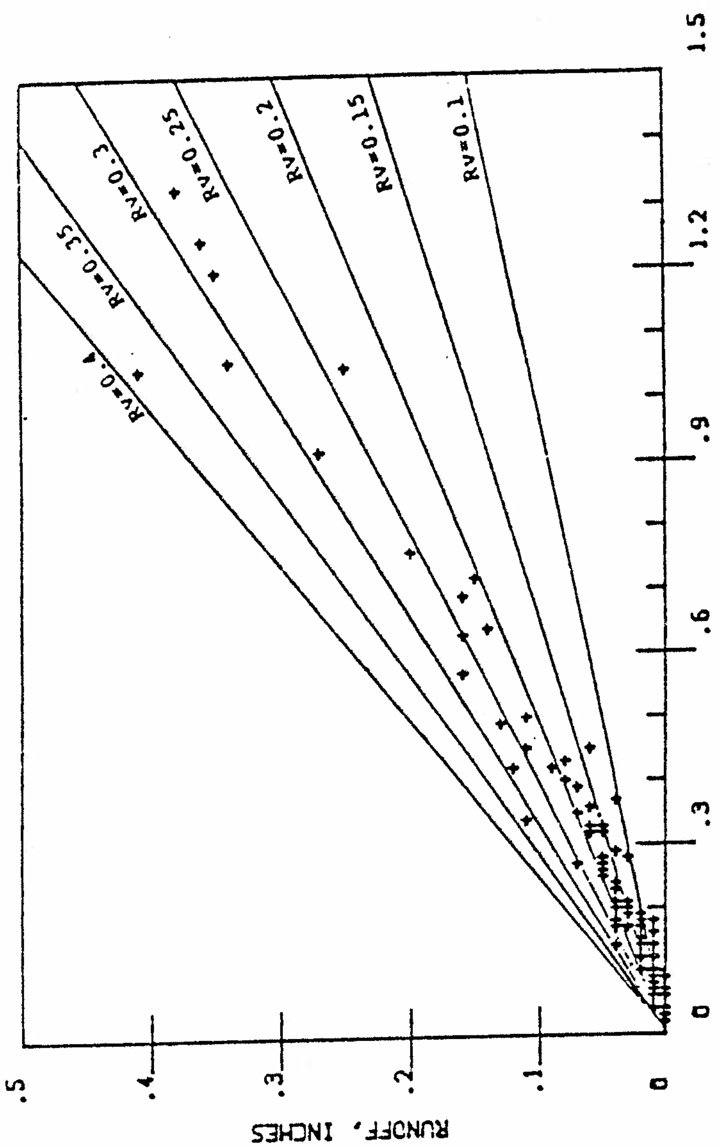
In order to use a hydrodynamic physical runoff model, a model representation for the urban area must be developed. Cascading planes are commonly used to route source area flows in urban areas using these procedures. The nonlinear dynamic response of the watershed is then portrayed using kinematic wave theory, a differential equation of continuity, and an approximation of the momentum equation. The nonlinear aspects of these models require many recalculations a: they account for varying conditions during the runoff event. The differential equations also need to be solved for different physical watershed configurations.

N. Recent Urban Hydrology Observations

Recent urban runoff studies have resulted in many runoff flow observations at many locations. These data have been analyzed to investigate the sources of urban runoff pollutants. A NURP (Nationwide Urban Runoff Program) study in Castro Valley, California, examined more than 60 runoff events during the 1979 and 1980 rain years (Pitt and Shawley 1982), and a study in Bellevue, Washington, collected runoff data from more than 400 storms from 1980 through 1982 (Pitt 1984). These data have been analyzed to estimate pollutant sources, but they all represent residential areas. Other NURP data has been collected nationwide (including Milwaukee), but has not been thoroughly evaluated. The West Coast NURP data indicated that the amount and character of runoff pollutants from a given area depend on rain variables (such as total rain depth, intensity, duration, and the interevent time between storms). Castro Valley had interevent periods of up to 100 days, while Bellevue had interevent periods of less than 20 days. These West Coast storms all had average rain intensities of about 1 mm/hr, wi 15 to 30 minute peak rain intensities of about 3 to mm/hr.

When the Bellevue data was analyzed, it was four that the runoff coefficients (Rv) varied substantiall by season and by total rain depth. Small rains had ve small Rv values (typically less than 0.1), while larg rains had Rv values approaching 0.5 (see Figure C.3). The average Rv values during the wet seasons were abo 35 percent larger than during the dry seasons when infiltration was greater. When multiple regression analyses were performed on this data, it was found th rainfall depth alone accounted for about 95 percent o the observed Rv values (when separated by season). Average rain intensities affected the Rv values by le than about 5 percent. Peak rain intensities accounted for between 5 and 10 percent of the Rv values. Increases in interevent periods reduced the Rv values by about 5 percent. Therefore, only season and total rain depth were found necessary in order to estimate and the total runoff volume with reasonable errors.

Figure C.3 Rainfall-Runoff Plot for Lake Hills (Bellevue, Wash.) Dry Season Data



Source: Pitt 1984

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