August 12, 2003

2. The Integration of Water Quality and Drainage Design Objectives

Introduction

Different drainage design criteria and receiving water use objectives often require the examination of different types of rains for the design of urban drainage systems. These different (and often conflicting) objectives of a stormwater drainage system can be addressed by using distinct portions of the long-term rainfall record. Several historical examinations (including Heaney, *et al*. 1977) have also considered the need for the examination of a wide range of rain events for drainage design. However, the lack of efficient computer resources severely restricted long-term analyses in the past. Currently, computer resources are much more available and are capable of much more comprehensive investigations (Gregory and James 1996). In addition to having more efficient computational resources, it is also necessary to re-examine some of the fundamental urban hydrology modeling assumptions (Pitt 1987). Most of the urban hydrology methods currently used for drainage design have been successfully used for large “design” storms. Obviously, this approach (providing urban areas safe from excessive flooding and associated flood related damages) is the most critical objective of urban drainage. However, it is now possible (and legally required in many areas) to provide urban drainage systems that also minimizes other problems associated with urban stormwater. This broader set of urban drainage objectives requires a broader approach to drainage design, and the use of hydrology methods with different assumptions and simplifications.

Runoff volume is usually the most important hydrology parameter in water quality studies, while peak flow rate and time of concentration are usually the most important hydrologic parameters for flooding and drainage studies. The relationships between these different hydrologic parameters and rain parameters are significantly different for different classes of rains. Runoff models for water quality investigations should therefore be different than the runoff models for flooding and drainage investigations. Similarly, flooding and drainage investigations should normally not use a hydrology model developed for water quality investigations.

The importance of different areas in a watershed as pollutant sources is dependent on accurate hydrology predictions. One also need to know the variations of each source area’s importance for different rains. Many control practice designs also depend on inflow hydrology. If one incorrectly predicts the sources of pollutants or flows, then one will not get expected stormwater control benefits. This section briefly describes a method to accurately predict the sources of urban runoff source flows during important small rains. This method is fundamental to the Source Loading and Management Model (WinSLAMM) that can be used in conjunction with the SWMM model.

Most existing stormwater models incorrectly predict flows associated with small rains in urban areas. This is important because common small storms are responsible for most of the annual urban runoff discharge quantities throughout North America (EPA 1983, Pitt 1987). Most existing urban runoff models originated from drainage and flooding evaluation procedures that emphasized very large rains (several inches in depth). These large storms only contribute very small portions of the annual average discharges. Obviously, the pollutant shock loadings and habitat destruction caused by a large storm may create significant receiving water use impairments, but a number of years will be available for recovery before another massive rain occurs. However, moderate storms, occurring several times a year, are responsible for the majority of the pollutant discharges. The effects caused by these frequent discharges are mostly chronic in nature (such as contaminated sediment and frequent high flow rates) and the interevent periods are not long enough to allow the receiving water conditions to recover (Pitt and Bozeman 1982).

Simplifying the assumptions concerning runoff losses for impervious and pervious areas for small rains has little significance on the accuracy of the predictions of runoff volumes for large rains. These same assumptions, however, cause dramatically large errors when predicting runoff associated with small rains, the rains of most importance for water pollutant discharges. The significance of small rains as important pollutant generators is then missed and controls are then designed for wrong storms and wrong source areas. The hydrology prediction method described here is a simplified procedure used to predict runoff volumes from individual homogeneous areas for a wide variety of rains. It requires knowledge of certain development characteristics of the urban area.

Rainfall and Runoff Characteristics for Urban Areas

Actual stormwater characteristics that can be used to evaluate design procedures were evaluated by Pitt, et al. (1999), and is summarized in this section. That evaluation examined data obtained from the EPA's Nationwide Urban Runoff Program (EPA 1983), the EPA's Urban- Rainfall-Runoff-Quality Data Base (Heaney, *et al*. 1982), and from the Humber River portion of the Toronto Area Watershed Management Study (Pitt and McLean 1986). The Toronto area data were from two extensively monitored watersheds, a residential/commercial area and an industrial area. Most of the EPA's "Data Base" data is from 2 locations in Broward County, FL; 1 site in Dade County, FL; 2 sites in Salt Lake City, UT; and 2 sites in Seattle, WA. Most of the data were obtained during the 1970s. These sites had the best representation of data of interest for these analyses and the sites were well described. Parameters examined included simultaneous rainfall and runoff depths, plus peak rain and flow rates. The following plots were prepared using this data:

- runoff depth versus rainfall,
- volumetric runoff coefficient (Rv) versus rainfall,
- NRCS curve number (CN) versus rainfall, and
- ratio of reported peak flow/peak rainfall versus rainfall.

In a similar manner, information from the EPA's NURP program (EPA 1983) was also investigated. A wider variety of information was collected during NURP, enabling additional relationships examining stormwater quality. Most of the data is from 5 sites in Champaign, IL; 2 sites in Austin, TX; 5 sites in Irondequoit Bay, NY; 1 site in Rapid City, SD; plus additional observations from Tampa, FL, Winston Salem, NC, and Eugene and Springfield, OR. Most of this data were obtained during the early 1980s and was subjected to rigorous quality control. Besides the four plots listed above, the following plots were also constructed examining potential water quality concentration relationships:

- total suspended solids concentration versus rainfall,
- COD concentration versus rainfall,
- phosphorous concentration versus rainfall,
- lead concentration versus rainfall,
- peak flow/peak rain versus rainfall, and
- peak flow rate versus peak rain intensity.

These plots were constructed to examine stormwater design methods using actual monitored data. These data can be used to examine many typical assumptions concerning stormwater drainage design and stormwater quality. Figures 2-1 through 2-9 show example plots for the John South Basin, a single family residential area, monitored during the EPA's NURP project in Champaign-Urbana, IL. The basic rainfall versus runoff plots (Figure 2-1) were made to indicate the smoothness of this basic relationship. A large scatter instead of a smooth curve may indicate measurement errors or uneven rainfalls over the catchment, or highly variable infiltration characteristics (due to changing soil moisture before the different rains). As shown on these plots, the runoff depth increases with increasing rain. However, several plots do show substantial scatter, mostly for sites having relatively small runoff yields. In addition, in some cases, more runoff was observed than could be accounted for by the rain. Errors in these measurements may be significant and would vary for the different sites. The senior authors of this report were involved in several of the monitoring projects that are included in these analyses, and also served on EPA technical committees overseeing others. In addition, we have many years experience in monitoring these parameters in many locations and recognize many of the past problems and current attempts to correct them. The following list therefore shows possible measurement errors that may have affected this data:

- variable rainfall over a large test catchment that was not well represented by enough rain gages (Although several of the test catchments had multiple rain gages, most did not, and few were probably frequently re-calibrated in the field.),
- poorly calibrated monitoring equipment (Many flow monitoring equipment relied on using the Manning's equation in pipes, with assumed roughness coefficients, without independent calibration, while other monitoring locations used calibrated insert weirs.)
- transcription errors (Many of these older monitoring activities required manual transfer from field equipment recorders to computers for analysis. In many cases, obvious "factor of ten" errors were made, for example.),
- newly developed equipment that has not been adequately tested, and
- difficult locations in the sewerage or streams that were monitored.

It is expected that the measurement errors were probably no less than about 25% during these monitoring activities. The effects of actual influencing factors can only be determined after the effects of these errors are considered.

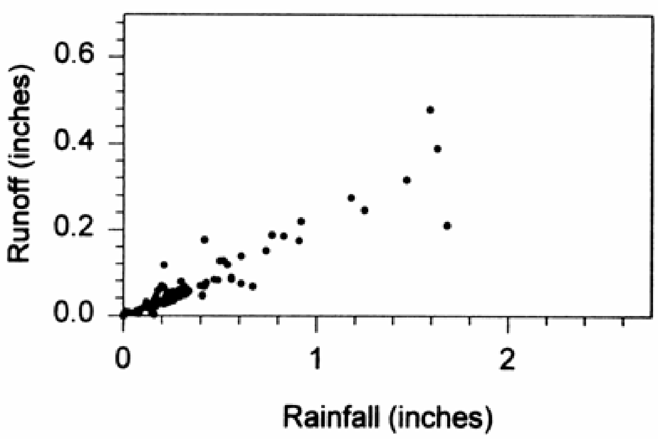

Figure 2-1. Runoff vs. rainfall.

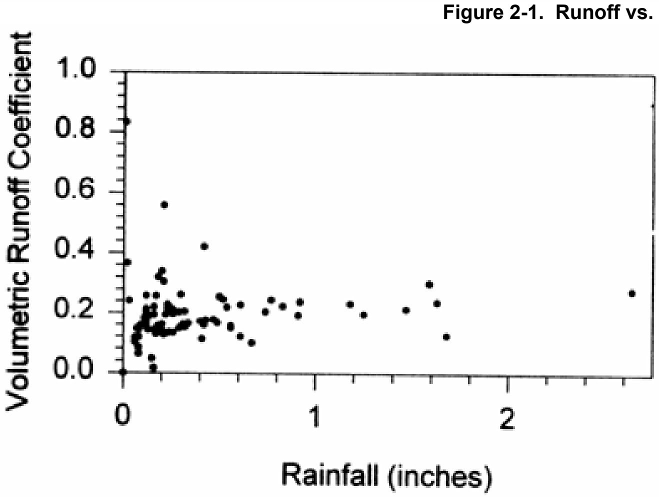

Figure 2-2. Rv vs. rainfall.

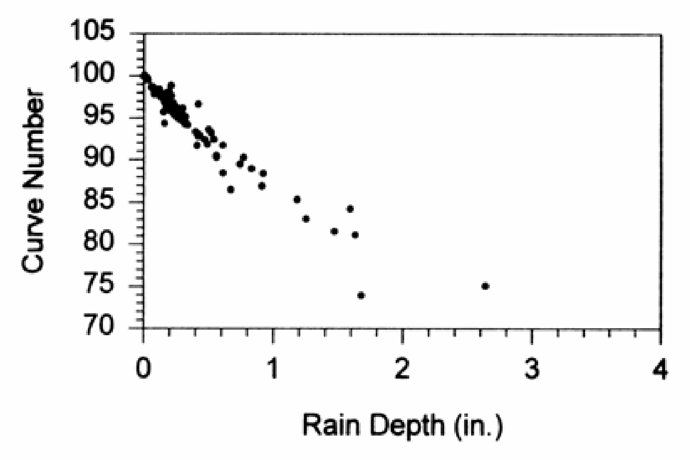

Figure 2-3. Curve number vs. rain depth.

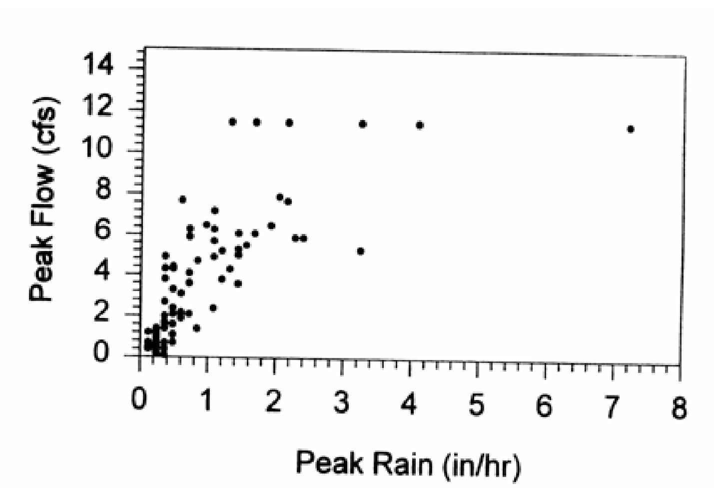

Figure 2-4. Peak flow vs. peak rain.

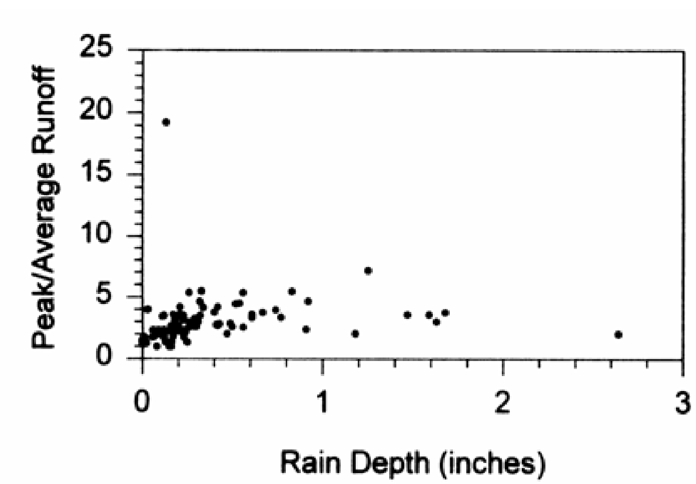

Figure 2-5. Peak/avg. runoff vs. rain depth.

Figure 2-6. SS vs. rain depth.

Figure 2-7. COD vs. rain depth.

Figure 2-8. Phosphorus vs. rain depth.

Figure 2-9. Lead vs. rain depth.

The plots of rainfall versus the volumetric runoff coefficient plot (Figure 2-2) shows the ratio of the runoff volume, expressed as depth for the watershed, to rain depth, or the Rv, for different rain depths. This is a related plot to the one described above. If the Rv ratio was constant for all events, the rainfall versus runoff depth plot described above, would indicate a straight diagonal line, with no scatter. It is typically assumed that the above described relationship would indicate increasing Rv values as the rain depth increased. Figure 2-1 shows a slight upwards curve with increasing rain depths. This is due to the rainfall losses making up smaller and smaller portions of the total rainfall as the rainfall increases, with a larger fraction of the rainfall occurring as runoff. The plot of Rv versus rainfall (Figure 2-2) would therefore show an increasing trend with increasing rain depth. In most cases, the plots of actual data indicate a large (random?) scatter, making the identification of a trend problematic. The use of a constant Rv for all rains may also be a problem because of the large scatter. In many cases, the long-term average Rv for a residential area may be close to the typically used value. In Figure 2-2, the values appear to center about 0.2 (somewhat smaller than the typically used value of about 0.3 for medium density residential areas), but the observed Rv values may range from lows of less than 0.04 to highs of greater than 0.5, especially for the smallest rains. The small rains probably have the greatest measurement errors, as the rainfall is much more variable for small rains than for larger rains, plus very low flows are difficult to accurately measure. Obviously, understanding what may be causing this scatter is of great interest, but is difficult because of measurement errors masking trends that may be present. In many cases, using a probability distribution to describe this variation may be the best approach.

Figure 2-3 is a plot of the NRCS curve number (CN) versus rainfall depth (SCS 1986). The NRCS assumes that the CN is constant for all rain depths for a specific site. However, they specify several limitations, including:

- the CN method is less accurate when the runoff is less than 0.5 inch. It is suggested that an independent procedure be used for confirmation,
- the CN needs to be modified according to antecedent conditions, especially soil moisture before an event, and
- the effects of impervious modifications (especially if they are not directly connected to the drainage path) needs to be reflected in the CN.

Few of these warnings are considered by most storm drainage designers, or by users of NRCS CN procedures for stormwater quality analyses. Figure 2-3 shows the typical pattern obtained when plotting CN against rain depth. The CN for small rain depths is always very large (approaching 100), then it decreases as the rain depth increases. At some point, the observed CN values equal the NRCS published recommended CN. During rains smaller than this matching point, the actual CN is greater than the NRCS CN. Predicted runoff depths would therefore be much less than the observed depths during these rains. Very large differences in runoff depths are associated with small differences in CN values, making this variation very important.

Figure 2-4 shows the observed peak runoff flow rate versus the peak rain intensity. If the averaging period for the peak flows and peak rain intensities were close to the catchment time of concentration (t_c), the slope of this relationship would be comparable to the Rational coefficient (C). The averaging times for the peak values probably ranged from 5 minutes to 1 hour for the different projects. Unfortunately, this averaging time period was rarely specified in the data documentation. Most urban area t_c values probably range from about 5 to 15 minutes. As indicated in this figure, the relationship between these two parameters shows a general upward trend, but it would be difficult to fit a statistically valid straight line through the data. As noted above for the other two drainage design procedures, actual real-world variations (coupled to measurement errors) add a lot of variation to the predicted runoff flow and volume estimates. Most drainage designers do not consider the actual variations that may occur.

Figure 2-5 shows an example plot of the ratio of the peak runoff flow rate to the average runoff flow rate versus rain depth. These values can be used to help describe the shape of simple urban area hydrographs. If the hydrograph can be represented by a simple triangular hydrograph, then the peak flow to average flow ratio must be close to 2. As shown on these figures, this ratio is typically substantially larger than 2 (it can never be less than 1 obviously), indicating the need to use a somewhat more sophisticated hydrograph shape (such as a double triangular hydrograph that can consider greater flows). These plots indicate if this ratio can be predicted as a function of rain depth. In most cases, values close to 2 are seen for the smallest rains, but they ratio increases to 5, or more, fairly quickly, but with much variability.

Example plots for total suspended solids, COD, phosphorous, and lead are shown on Figures 2-6 through 2-9 for each NURP site. It is commonly assumed that runoff pollutant concentrations are high for small rains (and at the beginning of all rains) and then taper off (the "first-flush" effect). As indicated on these plots, concentration has a generally random pattern. In many cases, the highest concentrations observed will occur for small events, but there is a large variation in observed concentrations at all rain depths. The upper limits of observed concentrations may show a declining curve with increasing rain depths, but the concentrations may best be described with random probability distributions. Analyses of concentrations versus antecedent dry periods can reduce some of this variability, as can analyses of runoff concentrations from isolated source areas.

Small Storm Hydrology

Stormwater Receiving Water Problems

Reviews of numerous urban receiving water studies from throughout the U.S. have identified the following diverse list of receiving water problems that may be caused by stormwater (Pitt 1995):

- Sedimentation damage in stormwater conveyance systems and in receiving waters.
- Nuisance algae growths from nutrient discharges into quiescent waters.
- Inedible fish and undrinkable water caused by toxic pollutant discharges.
- Shifts to less sensitive aquatic organisms caused by contaminated sediments and habitat destruction.
- Property damage from increased drainage system failures.
- Swimming beach closures from pathogenic microorganisms.
- Water quality violations, especially for bacteria and total recoverable heavy metals.

The first four problem areas are mostly associated with slug (mass) discharges (not instantaneous concentrations or rates), while the last three are mostly associated with instantaneous concentrations and high flow rates.

In order to predict receiving water problems caused by stormwater, accurate flow estimates and pollutant mass discharges must be known. Knowing where the potentially problem pollutants originate in the watershed is also valuable in order to select appropriate stormwater control candidates. Accurate knowledge of runoff volumes during different storms has been shown to be necessary when predicting pollutant discharges.

Typical Problems with Assumptions Commonly Used in Urban Hydrology Analyses

Most of the Annual Rain is Associated With Many Small Individual Events

This discussion reviews actual monitored rainfall and runoff distributions for Milwaukee, WI (data from Bannerman, *et al*. 1983), and examines long-term rainfall histories and predicted runoff from 24 locations throughout the U.S. The Milwaukee observations show that southeastern Wisconsin rainfall distributions can be divided into the following categories, with possible management approaches relevant for each category of rain:

- Common rains having relatively low pollutant discharges are associated with rains less than about 0.5 in. (12 mm) in depth. These are key rains when runoff-associated water quality violations, such as for bacteria, are of concern. In most areas, runoff from these rains should be totally captured and either re-used for on-site beneficial uses or infiltrated in upland areas. For most areas, the runoff from these rains can be relatively easily removed from the surface drainage system.

- Rains between 0.5 and 1.5 in. (12 and 38 mm) are responsible for about 75% of the runoff pollutant discharges and are key rains when addressing mass pollutant discharges. The small rains in this category can also be removed from the drainage system and the runoff re-used on site for beneficial uses or infiltrated to replenish the lost groundwater infiltration associated with urbanization. The runoff from the larger rains should be treated to prevent pollutant discharges from entering the receiving waters.

- Rains greater than 1.5 in. (38 mm) are associated with drainage design and are only responsible for relatively small portions of the annual pollutant discharges. Typical storm drainage design events fall in the upper portion of this category. Extensive pollution control designed for these events would be very costly, especially considering the relatively small portion of the annual runoff associated with the events. However, discharge rate reductions are important to reduce habitat problems in the receiving waters. The infiltration and other treatment controls used to handle the smaller storms in the above categories would have some benefit in reducing pollutant discharges during these larger, rarer storms.

- In addition, extremely large rains also infrequently occur that exceed the capacity of the drainage system and cause local flooding. Two of these extreme events were monitored in Milwaukee during the Nationwide Urban Runoff Program (NURP) project (EPA 1983). These storms, while very destructive, are sufficiently rare that the resulting environmental problems do not justify the massive stormwater quality controls that would be necessary for their reduction. The problem during these events is massive property damage and possible loss of life. These rains typically greatly exceed the capacities of the storm drainage systems, causing extensive flooding. It is critical that these excessive flows be conveyed in "secondary" drainage systems. These secondary systems would normally be graded large depressions between buildings that would direct the water away from the buildings and critical transportation routes and to possible infrequent/temporary detention areas (such as large playing fields or parking lots). Because these events are so rare, institutional memory often fails and development is allowed in areas that are not indicated on conventional flood maps, but would suffer critical flood damage.

Obviously, the critical values defining these rain categories are highly dependent on local rain and development conditions. Computer modeling analyses from several representative urban locations from throughout the U.S. are presented in this paper. These modeled plots indicate how these rainfall and runoff probability distributions can be used for more effective storm drainage design in the future. In all cases, better integration of stormwater quality and drainage design objectives will require the use of long-term continuous simulations of alternative drainage designs in conjunction with upland and end-of-pipe stormwater quality controls. The complexity of most receiving water quality problems prevents a simple analysis. The use of simple design storms, which was a major breakthrough in effective drainage design more than 100 years ago, is not adequate when receiving water quality issues must also be addressed.

This discussion also reviews typical urban hydrology methods and discusses common problems in their use in predicting flows from these important small and moderate sized storms. A general model is then described, and validation data presented, showing better runoff volume predictions possible for a wide range of rain conditions.

Figure 2-10 includes cumulative probability density functions (CDFs) of measured rain and runoff distributions for Milwaukee during the 1981 NURP monitored rain year (data from Bannerman, *et al*. 1983). CDFs are used for plotting because they clearly show the ranges of rain depths responsible for most of the runoff. Rains between 0.05 and 5 in. were monitored during this period, with two very large events (greater than 3 inches) occurred during this monitoring period which greatly distort these curves, compared to typical rain years. The following observations are evident:

- The median rain depth was about 0.3 in.
- 66% of all Milwaukee rains are less than 0.5 in. in depth.
- For medium density residential areas, 50% of runoff was associated with rains less than 0.75 in.
- A 100-yr., 24-hr rain of 5.6 in. for Milwaukee could produce about 15% of the typical annual runoff volume, but it only contributes about 0.15% of the average annual runoff volume, when amortized over 100 yrs.
- Similarly, a 25-yr., 24-hr rain of 4.4 in. for Milwaukee could produce about 12.5% of the typical annual runoff volume, but it only contributes about 0.5% of the average annual runoff volume, when amortized over 25 yrs.

Figure 2-11 shows CDFs of measured Milwaukee pollutant loads associated with different rain depths for a medium density residential area. Suspended solids, COD, lead, and phosphate loads are seen to closely follow the runoff volume CDF shown in Figure 2-10, as expected. Since load is the product of concentration and runoff volume, some of the high correlation shown between load and rain depth is obviously spurious. However, these overlays illustrate the range of rains associated with the greatest pollutant discharges.

The monitored rainfall and runoff distributions for Milwaukee show the following distinct rain categories:

- <0.5 inch. These rains account for most of the events, but little of the runoff volume, and are therefore easiest to control. They produce much less pollutant mass discharges and probably have less receiving water effects than other rains. However, the runoff pollutant concentrations likely exceed regulatory standards for several categories of critical pollutants, especially bacteria and some total recoverable metals. They also cause large numbers of overflow events in uncontrolled combined sewers. These rains are very common, occurring once or twice a week (accounting for about 60% of the total rainfall events and about 45% of the total runoff events that occurred), but they only account for about 20% of the annual runoff and pollutant discharges. Rains less than about 0.05 inches did not produce noticeable runoff.

- 0.5 to 1.5 inches. These rains account for the majority of the runoff volume (about 50% of the annual volume for this Milwaukee example) and produce moderate to high flows. They account for about 35% of the annual rain events, and about 20% of the annual runoff events. These rains occur on the average about every two weeks during the spring to fall seasons and subject the receiving waters to frequent high pollutant loads and moderate to high flows.

- 1.5 to 3 inches. These rains produce the most damaging flows, from a habitat destruction standpoint, and occur every several months (at least once or twice a year). These recurring high flows, which were historically associated with much less frequent rains, establish the energy gradient of the stream and cause unstable streambanks. Only about 2 percent of the rains are in this category and they are responsible for about 10 percent of the annual runoff and pollutant discharges.

Figure 2-10. Milwaukee rain and runoff distributions.

• >3 inches. This category is rarely represented in field studies due to the rarity of these large events and the typically short duration of most field observations. The smallest rains in this category are included in design storms used for drainage systems in Milwaukee. These rains occur only rarely (once every several years to once every several decades, or less frequently) and produce extremely large flows. The 3-year monitoring period during the Milwaukee NURP program (1980 through 1983) was unusual in that two of these events occurred. Less than 2 percent of the rains were in this category (typically <<1% would be), and they produced about 15% of the annual runoff quantity and pollutant discharges. During a “normal” period, these rains would only produce a very small fraction of the annual average discharges. However, when they do occur, great property and receiving water damage results. The receiving water damage (mostly associated with habitat destruction, sediment scouring, and the flushing of organisms great distances downstream and out of the system) can conceivably naturally recover to before-storm conditions within a few years.

Figure 2-11. Milwaukee pollutant discharge distributions.

These rainfall and pollutant mass distributions are not unique for Milwaukee. Long-term continuous simulations were made using WinSLAMM (incorporating the small storm hydrology components described in this report section) for 22 representative locations from throughout the U.S. (Figure 2-12). These locations represent most of the major river basins and much of the rainfall variations in the country. These analyses are only intended to show the importance of these smaller rains for many different regions and conditions. They are not intended to be used for design purposes. As noted earlier, the recommended approach for design is to continuously model long rain records for site specific conditions. These locally derived runoff distributions, reflecting site conditions and actual rains, can then used for evaluating alternative drainage and water quality designs.

These simulations were based on 5 to 10 years of rainfall records, usually containing about 500 individual rains. The rainfall records were from certified NOAA weather stations and were obtained from CD-ROMs distributed by EarthInfo of Boulder, CO. Hourly rainfall depths for the indicated periods were downloaded from the CD-ROMs into an Excel spreadsheet. The files were slightly modified (by eliminating the daily total rainfall column) and saved as a comma delineated file. This file was then read by an utility program included in the WinSLAMM package. This rainfall file utility combined adjacent hourly rainfall values into individual rains, based on user selections (at least 6 hrs of no rain was used to separate adjacent rain events and all rain depths were used, with the exception of the “trace” values). These rain files for each city were then used in WinSLAMM for typical medium density and strip commercial developments. The outputs of these computer runs were then plotted as shown on Figure 2-13.

Figure 2-12. U.S. major river basins and modeled cities.

Figure 2-13a. Modeled rain, runoff, and pollutant distributions.

Figure 2-13b. Modeled rain, runoff, and pollutant distributions (cont.).

Figure 2-13c. Modeled rain, runoff, and pollutant distributions (cont.).

Figure 2-13d. Modeled rain, runoff, and pollutant distributions (cont.).

Figure 2-13e. Modeled rain, runoff, and pollutant distributions (cont.).

Figure 2-13f. Modeled rain, runoff, and pollutant distributions (cont.).

Table 2-1 summarizes these rain and runoff distributions for different U.S. locations, while Figures 2-14 through 2-19 plot some of the important values on a U.S. map. Lower and upper runoff distribution breakpoints were identified on all of the individual distributions. The breakpoints separate the distributions into the following three general categories:

• less than lower breakpoint: small, but frequent rains. These generally account for 50 to 70 percent of all rain events (by number), but only produce about 10 to 20 percent of the runoff volume. Figure 2-15 shows that the rain depth for this breakpoint ranges from about 0.10 in. in the Southwest arid regions of the country, to about 0.5 in. in the wet Southeast. These events are most important because of their frequencies, not because of their mass discharges. These rains are therefore of great interest where water quality violations associated with urban stormwater occur. This would be most common for bacteria (especially fecal coliforms) and for total recoverable heavy metals which typically exceed receiving water numeric criteria during practically every rain event in heavily urbanized drainages having separate stormwater drainage systems.

• between the lower and upper breakpoint: moderate rains. These rains generally account for 30 to 50 percent of all rains events (by number), but produce 75 to 90 percent of all of the runoff volume (Figure 2-19). Figure 2-17 shows that the rain depths associated with the upper breakpoint range from about 1 to 2 in. in the arid parts of the U.S. to up to 5 or 6 in. in wetter areas. As shown earlier for actual monitored events in Milwaukee and elsewhere, these runoff volume distributions are approximately the same as the pollutant distributions. Therefore, these intermediate rains also account for most of the pollutant mass discharges and much of the actual receiving water problems associated with stormwater discharges.

• above the upper breakpoint: large, but rare rains. These rains include the typical drainage design events and are therefore quite rare. During the period analyzed, many of the sites only had one or two, if any, events above this breakpoint. These rare events do account for about 5 to 10 percent of the runoff on an annual basis, as shown on Figure 2-18. Obviously, these events must be evaluated to ensure adequate drainage.

Because of the importance of these small and moderate rains, it is important to review typically used urban hydrology methods that have been commonly used to predict runoff from urban areas. These tools have been reasonably successful when evaluating drainage capacity for large “design storm” events. However, the following paragraphs will indicate their short-comings when used for evaluating the common smaller events. A general urban runoff model is also presented that has been shown to be useful to predict runoff volumes for a wide range of rain events, especially the small and moderate rains of greatest interest in water quality evaluations.

The Rainfall-Runoff Inter-Relationships for Different Urban Areas are Surprisingly Similar

Figure 2-20 shows a dendogram from a cluster analysis (using SYSTAT) of rainfall and runoff data from two areas: an industrial area and a residential and commercial mixed land use area (Pitt 1987). Most of the variation in runoff volumes for different rains can be explained by rain volume variations alone. Rain intensity and antecedent periods are not very important when predicting runoff volumes. However, rain intensity information is very important for predicting runoff rates which are needed for drainage and flooding studies. It is also noted that the runoff duration is closely related to rain duration. A simple procedure for predicting runoff volume is possible using only total rain depth (and land development characteristics).

Figure 2-14. Median rain depth (in.).

Figure 2-15. Lower breakpoint rain depth (in.).

Figure 2-16. Percentage of rain events less than lower breakpoint.

Figure 2-17. Upper breakpoint rain depth (in.).

Figure 2-18. Percentage of runoff volume greater than upper breakpoint.

Figure 2-19. Percentage of runoff volume between breakpoints.

Figure 2-20. Cluster analysis (dendogram) for basic urban hydrology structure (Pitt 1987).

Varying Contributing areas are Important in Urban Hydrology

Figure 2-21 shows the components of a hypothetical hydrograph for an urban area. For small rains, most of the runoff observed at the outfall originates from street surfaces and other directly connected impervious areas. However, as the rain depth increases, runoff from pervious areas become important. The critical problem is being able to predict when these component areas contribute significant runoff volumes (and pollutants). WinSLAMM (Pitt 1986 and 1992) was developed to enable predictions of runoff contributions (and source area controls), using a simplified urban hydrology approach appropriate for important small rains.

Figure 2-21. Variable contributing area unit hydrographs for urban site.

Observed Runoff Volumes Do Not Compare Well With Commonly Used Urban Runoff Prediction Methods

Some of the most commonly used stormwater design methods utilizes the NRCS curve number (CN) method, especially TR-20 and TR-55 (SCS 1986). The NRCS recommends against the use of the curve number procedure for rains less than one-half inch. Unfortunately, this warning is ignored in many urban runoff models that have been developed. As shown previously, small rains are very significant when analyzing urban runoff. In addition, the NRCS recommends that the curve number method should be used for individual components of the drainage area, if CN values differ by more than 5, instead of using a composite CN for the complete area. Unfortunately, many users of the CN method ignore these two basic warnings, and many urban stormwater models use composite CN values for all storms. The CN method is a suitable tool if properly used, unfortunately, it is frequently used for small storms and for water quality evaluations, well beyond its intended use addressing drainage design for conveyance objectives for large rains.

Figure 2-22a shows rainfall-runoff plots for eight monitored areas in Milwaukee. The curve is similar to the US. Natural Resources Conservation Service (NRCS) curve number (CN) rainfall-runoff plot contained in TR-55 (SCS 1986). This figure also shows the NRCS CN values calculated using actual P (precipitation) and Q (runoff quantity) data. CNs vary greatly with rain depth.

Figure 2-22b shows that CNs at the Milwaukee NURP monitored sites did not approach the published CN values for typical medium density residential areas until the rains were much greater than five inches. The Milwaukee high density land use areas can use published CN values for rains as small as two inches, while the Milwaukee commercial area CNs are correct when close to one inch.

Pitt, *et al*. 1999 shows numerous similar plots for other monitored locations from throughout the U.S., collected during the EPA's NURP projects in the early 1980s (EPA 1983), and from the EPA's rainfall-runoff-quality data base (Huber, *et al.* 1982). Figures 2-23 through 2-26 contain CN versus rain depth plots for many of these cities, including: 2 locations in Broward County, FL; 1 site in Dade County, FL; 2 sites in Salt Lake City, UT; and 2 sites in Seattle, WA (from the rainfall-runoff-quality data base), plus 4 sites in Champaign, IL; 5 sites in Irondequoit Bay, NY; 2 sites in Austin, TX; and 1 site in Rapid City, SD (from the NURP data). Figure 2-23 contains plots for areas with little urbanization, Figure 2-24 contains plots for medium density residential areas and mixed common urban areas, Figure 2-25 contains plots for high density and commercial areas, and Figure 2-26 contains plots for catchments having only major roadways. In all cases, the general pattern is the same: observed curve numbers are all very high for small rains, tapering off as the rains become large. All of the test watersheds are typical for these land uses and do not contain any unusual drainage designs or stormwater controls.

Table 2-2 is a summary of these observed curve numbers at several different rain depths, compared to typical curve numbers presented by the NRCS (SCS 1986) for these land uses. Several of the sites had adequate descriptions to enable curve numbers to be estimated, based on their directly connected impervious areas and soil texture. The following list shows these sites, with the NRCS recommended curve numbers, and the approximate rain depth where these curve numbers were observed:

- Broward Co., FL, residential land use (40% imperv., with sandy soils). NRCS CN = 61, observed at about 3.5 in. of rain.

- Champaign-Urbana, IL, single family residential land use (18% imperv., with silty, poorly drained soils). NRCS CN = 84, observed at about 1.2 in. of rain.

- Champaign-Urbana, IL, single family residential land use (19% imperv., with silty, poorly drained soils). NRCS CN = 84, observed at about 1.2 in. of rain.

- Dade Co., FL, high density residential land use (almost all impervious, "D" soils). NRCS CN = 92, observed at about 1.3 in. of rain.

Table 2-1. Rainfall and Runoff Distribution Characteristics for Different Locations from Throughout the U.S.

	Median rain depth, by count (in)	Corresponding percentage of runoff for the median rain depth	Rain depth associated with median runoff depth (in)	Lower breakpoint rain depth (in)	Percentage of rain events less than lower breakpoint	Percentage of runoff volume less than lower breakpoint	Upper breakpoint rain depth (in)	Percentage of rain events less than upper breakpoint	Percentage of runoff volume less than upper breakpoint	Percentage of runoff volume between breakpoints	Percentage of rain events between breakpoints
lumbia rth cific											
ise, ID	0.07	3 - 5	0.30 – 0.35	0.10	52	9 - 11	0.91	99	89 - 93	80 - 82	47
attle, WA	0.12	4 - 6	0.62 – 0.80	0.18	60	8 - 11	3.4	99	92 - 96	84 - 85	39
lifornia											
s geles, CA	0.18	3 - 5	1.2 – 1.5	0.29	64	7 - 10	3.5	99	92 - 98	85 - 88	35
eat Basin no, NV	0.07	3 - 5	0.35 – 0.41	0.10	61	8 - 10	1.7	99	93 - 95	85	38
wer lorado oenix, AZ	0.10	4 - 6	0.55 – 0.68	0.19	64	9 - 12	2.3	99	94 - 98	85 - 87	35
ssouri ings, MT	0.06	2 - 4	0.55 – 0.60	0.12	64	8 - 10	1.6	99	89 - 93	81 - 83	35
nver, CO	0.08	2 - 4	0.50 – 0.60	0.19	71	13 - 17	1.8	99	91 - 95	78	28
pid City,	0.06	2 - 4	0.50 – 0.55	0.15	69	10 - 13	1.9	99	92 - 96	82 - 83	30
kansas- iite-Red chita, KS	0.13	2 - 5	1.1 – 1.4	0.31	65	10 - 13	3.0	99	88 - 93	78 - 80	34
kas Gulf stin, TX	0.14	2 – 3	1.4 – 1.8	0.50	72	8 - 12	6.0	99	88 - 94	80 - 82	27
per ssissippi nneapolis, I	0.11	3 - 5	0.73 – 1.0	0.22	65	9 - 13	2.8	99	94 - 96	83 - 85	34
dison, WI	0.12	3 - 5	0.78 – 0.98	0.23	65	9 - 13	3.5	99	97 - 99	86 - 88	34
waukee,	0.12	2 - 4	0.9 – 1.1	0.25	65	9 - 12	2.5	99	89 - 95	80 - 83	34
Louis,)	0.14	4 - 6	1.0 – 1.2	0.31	65	10 - 13	2.8	99	90 - 95	80 - 82	34
eat kes troit, MI	0.20	7 - 11	0.72 – 0.81	0.20	50	7 - 11	2.4	99	92 - 95	85 - 84	49
ffalo, NY	0.11	2 - 4	0.61 – 0.72	0.12	64	8 - 12	2.1	99	88 - 93	80 - 81	35
io lumbus, I	0.12	3 - 5	0.80 – 1.0	0.22	63	8 - 12	2.2	99	85 - 91	77 - 79	36

rth Atlantic											
rtland, ME	0.15	2 - 4	1.1 – 1.5	0.30	64	8 - 12	4.5	99	90 - 96	82 - 84	35
wark, NJ	0.28	6 - 12	1.2 – 1.5	0.33	54	8 - 12	3.3	99	89 - 94	81 - 82	45
wer Mississippi											
w Orleans, LA	0.25	3 - 5	1.7 – 2.2	0.45	62	7 - 11	4.0	99	88 - 93	81 - 82	37
uth Atlantic lf											
anta, GA	0.22	3 – 5	1.2 – 1.7	0.32	58	5 – 9	4.0	99	91 – 95	86	41
mingham, AL	0.20	3 - 5	1.2 – 1.5	0.40	64	8 - 13	5.0	99	90 - 96	82 - 83	35
leigh, NC	0.18	4 - 6	1.0 – 1.2	0.26	60	7 - 11	2.5	99	87 - 93	80 - 82	39
ami, FL	0.13	3 - 5	1.2 – 1.6	0.30	67	9 - 13	4.0	99	87 - 93	78 - 80	32

Figure 2-22. Observed rainfall-runoff and curve numbers for Milwaukee (Pitt 1987).

- Champaign-Urbana, IL, commercial land use (40% imperv., with silty and poorly drained soils). NRCS CN = 87, observed at about 1.1 in. of rain.
- Champaign-Urbana, IL, commercial land use (55% imperv., with silty and poorly drained soils). NRCS CN = 91, observed at about 0.8 in. of rain.
- Broward Co., FL, transportation catchment (54% imperv., with sandy soils). NRCS CN = 73, observed at about 1.7 in. of rain.
- Salt Lake City, UT, roadway land use (mostly paved, sandy loam). NRCS CN = 89, observed at about 0.3 in. of rain.

- Salt Lake City, UT, transportation catchment (imperv. Raods, clay loam). NRCS CN = 95, observed at about 0.15 in. of rain.

For the rains less than the matching point (rain depth where the NRCS recommended CN was observed), the actual CN is larger than the recommended CN and the predicted runoff using the NRCS methods would be less than actually occurred. Similarly, for rains larger than the matching point, the actual CN is smaller than the recommended CN and the predicted runoff using the NRCS CN method would be greater than actually occurred. The magnitude of the runoff differences varies greatly, depending on the CN values and the rain depth. As an example, if the recommended NRCS CN was 84, but the actual CN was really 98 for a 0.2 in. rain (similar to the Champaign, IL, medium density residential sites), the percentage error is infinite. For a 1 in. rain, the actual CN at this site was about 86 and the recommended NRCS value remains at 84. The difference now is much smaller, as the rain depth being examined is close to the matching point depth of 1.2 inches. If the rain depth of concern was much larger, say 3 inches, the errors would be in the other direction, as summarized below:

	0.2 in. rain (matching point of 1.2 in)	1 in. rain (matching point of 1.2 in)	3 in. rain (matching point of 1.2 in)
CN of 84 (recommended by NRCS)	0 in. of runoff predicted by NRCS	0.15 in. of runoff predicted by NRCS	1.52 in. of runoff predicted by NRCS
Actual CN and predicted runoff	0.10 in. of runoff observed (actual CN of 98)	0.20 in. of runoff observed (actual CN of 86)	0.91 in. of runoff observed (actual CN of 74)
	Actual is infinitely larger, predicted is infinitely less.	Actual is larger, predicted is less. Error of 25%.	Actual is less, predicted is larger. Error of –67%.

The overall annual runoff depth error associated with using the NRCS recommended CN method depends on the frequency of rains having the different errors. Because the matching point rainfall depths are close to the rain depth associated with the median runoff depth, as shown previously on 2-1, the annual errors may be within reason. However, the errors associated with individual events, and for the three classes of rain depths described earlier, are likely very large. This is a significant problem with stormwater quality management where accurate representations of the sources of the runoff are needed in order to evaluate control practices and development options. If the relative sources of the runoff flows are in great error, inappropriate and wasteful expenditures are likely.

Figure 2-23. Low density development observed CN vs. rain depth plots.

Figure 2-24. Medium density land use area observed CN vs. rain depth plots.

Figure 2-25. High density residential and commercial area observed CN vs. rain depth plots.

Figure 2-26. Transportation land use area observed CN vs. rain depth plots.

Table 2-2. Observed Curve Numbers Compared to Typically Used Values

Land Use and Location	Directly connected imperviousness	0.2 in. rain	0.5 in. rain	1 in. rain	3 in. rain	For max. rain observed	Estimated CN from NRCS tables for different soil conditions (if possible, most likely CN highlighted, based on available site description):			
Low Density/Suburban							A (sandy to sandy loam)	B (silt loam or loam)	C (sandy clay loam)	D (silty to clayey)
Austin, TX	21%	94	84	72	53	42 (5 in.)	51	68	79	84
Irondequoit Bay, NY	Rv = 0.1	95	88	76	55	52 (4 in.)	46	65	77	82
Irondequoit Bay, NY	Rv = 0.2	94	86	77	57	52 (4 in.)	51	68	79	84
Irondequoit Bay, NY	Rv = 0.2	94	89	84	69	67 (4 in.)	51	68	79	84
Medium Density Residential										
Austin, TX	39%	96	89	82	66	52 (5 in.)	61	75	83	87
Broward County, FL	40% (sandy soils)	96	89	81	65	54 (5 in.)	**61**	75	83	87
Champaign-Urbana, IL	18% (silty, poorly drained soils)	96	94	87	72	71 (4 in.)	51	68	79	**84**
Champaign-Urbana, IL	19 % (silty, poorly drained soils)	98	93	86	74	72 (4 in.)	51	68	79	**84**
Rapid City, SD	mixed	95	92	84	67	63 (4 in.)	?	?	?	?
High Density Residential										
Dade County, FL	"Almost all imperv." (D soils)	99	97	94	87	82 (7 in.)	77	85	90	**92**
Seattle, WA	?	94	89	80	56 (max.)		77	85	90	92

Commercial										
Champaign-Urbana	40% (silty, poorly drained soils)	97	95	89	81 (max.)		61	75	83	**87**
Champaign-Urbana	55% (silty, poorly drained soils)	99	95	89	74	73 (4 in.)	73	82	88	**91**
Seattle, WA	?	90	76	61	44 (max.)		?	?	?	?
Irondequoit Bay, NY	?	92	82	72	46	46 (4 in.)	?	?	?	?
Transportation										
Broward County, FL	54% (sandy soils)	96	93	86	62	53 (5 in.)	**73**	82	88	91
Salt Lake City, UT	Mostly paved (sandy loam)	91	81	67	na	na	**89**	92	94	95
Salt Lake City, UT	"imperv. roads" (clay loam)	95	84	73	na	na	89	92	94	**95**

Actual Volumetric Runoff Coefficients (Rv) Vary With Storm Size.
Figure 2-27 shows how the volumetric runoff coefficients (the ratio of runoff depth to rainfall depth) change with rain depth. After subtracting initial abstractions, continuous losses can be assumed to be mostly infiltration. After a sufficient amount of rain has occurred, all losses have been satisfied. Each unit increase in rain then results in a unit increase in runoff volume.

Small rain depths result in runoff that have small Rv values. As the rain depth increases, the Rv increases. Rv values are only "constant" over a small range in rain depths. During many urban runoff monitoring projects, only small ranges of rains are typically represented. Therefore, "averaged" Rv values are incorrectly used with the understanding that they are useful over a wider range than justified. The NURP data was collected in the early 1980s, while the rainfall-runoff-quality data base information was collected much earlier. There was significant variation in the accuracies of monitoring rainfall and runoff for the different locations. This is most evident at test sites having large amounts of directly connected pavement. Many of the measured runoff events had greater runoff volumes than the measured rainfall volumes (Rv values greater than 1.0 and calculated CN values greater than 100). This of course cannot occur in the absence of other flow sources and was likely associated with random measurement errors. The best measurements were probably made with errors approaching 25%, while some test sites used newly available equipment and errors may have been greater. These errors are much more obvious at high density and commercial sites than at the more commonly monitored medium density residential sites.

Figure 2-27. Rainfall-runoff plot showing losses and Rv values (Pitt 1987).

Figure 2-28. Idealized plots of Rv and CN values.

Figure 2-28 shows a plot of runoff depth versus rain depth and another plot of the NRCS CN versus rain depth for a set of artificial rain and runoff data. These plots were prepared to visually show the relationship between Rv and NRCS CN values. If the data has relatively constant Rv values for all rains, the CN plots will naturally decrease substantially with increasing rain depth (again, as indicated in almost all of the measured data). It is interesting to note that the calculated NRCS CN is always very close to 100 for very small rain and runoff values, irrespective of the Rv ratio. The Rv values likely increase with increasing rain depth, which is evident if the observations can be obtained with small measurement errors and if the range of rains observed is large. Flow and rainfall measurement errors are much more obvious on the Rv plots, especially for the small rains, than on the CN plots.

Small Storm Hydrology Model

Runoff Process for Paved Surfaces

When rain falls on an impervious surface, much of it will flow off the surface and contribute to the total urban runoff. With the exception of infiltration, these losses are mostly associated with the initial portions of the rain and are termed initial abstractions. Water may also infiltrate through pavement, or through cracks or seams in the pavement. For small rains, a much greater portion of the rain will be lost to these runoff loss processes than for large rains.

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) found that smooth paved surfaces had lower infiltration losses, compared to poorly maintained surfaces which had losses of about 7 percent of the total rain. 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. 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) measured infiltration rates through typical “sealed” seams of about 20 mm per hour (with pavement seams located about every 8 meters).

Infiltration of Rain Water Through Pavement Can be a Substantial Portion of the Total Rain for Most Events

Initial abstractions are dependent of pavement texture and slope, while infiltration is dependent on pavement porosity and pavement cracks. Pavement is relatively porous. It is the pavement base course that is much more resistant to percolation. Infiltrated water is therefore forced to flow laterally towards the pavement edges. If the flow path is long, then the resulting infiltration is limited. Figure 2-29 is an example from a typical pavement runoff test (Pitt 1987). Initial abstractions may be about 1 mm for pavement, while the total infiltration may be between 5 and 10 mm. The maximum losses may occur after about 20 mm of rain.

Variable Runoff Losses as a Function of Time Indicate Very Different Infiltration Values for Different Rain Intensities

Figure 2-30a shows that high infiltration rates are associated with high rainfall intensities (Pitt 1987). The Horton equation predicts a single infiltration relationship as a function of time, irrespective of rain intensity. When variable runoff losses are plotted against total rain depth (Figure 2-30b) a single relationship is seen (rain intensity multiplied by time duration gives rain depth). Horton actually recommended infiltration as a function of rain depth, but current practice of using double-ring infiltrometers to calibrate the Horton equation does not allow infiltration measurements to be made as a function of rain depth, only as a function of time for the ponded test conditions.

Infiltration in Disturbed Urban Soils

Disturbed Urban Soils Do Not Behave as Indicated by Typically Used Models

More rain infiltrates through pavement surfaces and less rain infiltrates through soils than typically assumed. Double-ring infiltrometer test results from Oconomowoc, WI, urban soils (Table 2-3) indicated highly variable infiltration rates for soils that were generally sandy (NRCS A/B hydrologic group soils). The median initial rate was about 3 in/hr, but ranged from 0 to 25 in/hr. The final rates also had a median value of about 3 in/hr after at least two hours of testing, but ranged from 0 to 15 in/hr. Many infiltration rates actually increased with time during these tests. In about 1/3 of the cases, the observed infiltration rates remained very close to zero, even for these sandy soils. Areas that experienced substantial disturbances or traffic (such as school playing fields) had the lowest infiltration rates, typically even lower than concrete or asphalt! These values indicate the large variability in infiltration rates that may occur in areas having supposedly similar soils. Obviously, these variations can significantly affect site specific runoff predictions. The lowest infiltration rates were observed in areas having heavy foot traffic and in areas obviously impacted by silt, while the highest rates were in relatively undisturbed areas.

Table 2-3. Ranked Oconomowoc, WI, Double Ring Infiltration Test Results

Observed urban soil Infiltration rates (in/hr):

Initial Rate	Final Rate (after 2 hours)	Total Observed Rate Range
25	15	11 to 25
22	17	17 to 24
14.7	9.4	9.4 to 17
5.8	9.4	0.2 to 9.4
5.7	9.4	5.1 to 9.6
4.7	3.6	3.1 to 6.3
4.1	6.8	2.9 to 6.8
3.1	3.3	2.4 to 3.8
2.6	2.5	1.6 to 2.6
0.3	0.1	<0.1 to 0.3
0.3	1.7	0.3 to 3.2
0.2	<0.1	<0.1 to 0.2
<0.1	0.6	<0.1 to 0.6
<0.1	<0.1	all <0.1
<0.1	<0.1	all <0.1
<0.1	<0.1	all <0.1

In an attempt to explain much of the variation shown in the above early tests, recent tests of infiltration through disturbed urban soils were conducted in the Birmingham, AL, area by the author and UAB students. Eight categories of soils were tested, with about 15 to 20 individual tests conducted in each of eight categories (comprising a full factorial experiment). Numerous replicates were needed in each category because of the expected high variation in infiltration rates. The eight categories tested were as follows:

Category	Soil Texture	Compaction	Moisture
1	Sand	Compact	Saturated
2	Sand	Compact	Dry
3	Sand	Non-compact	Saturated
4	Sand	Non-compact	Dry
5	Clay	Compact	Saturated
6	Clay	Compact	Dry
7	Clay	Non-compact	Saturated
8	Clay	Non-compact	Dry

Figure 2-31 contains plots showing the interactions of moisture and compaction on infiltration for both soil texture conditions. Four general conditions were observed to be statistically unique:

- noncompact sandy soils
- compact sandy soils
- noncompact and dry clayey soils
- all other clayey soils

Compaction has the greatest effect on infiltration rates in sandy soils, with little detrimental effects associated with soil moisture. Clay soils, however, are affected by both compaction and moisture. Compaction is seen to have about the same effect as moisture on these soils, with saturated and compacted clayey soils having very little effective infiltration. In most cases, the mapped soils were similar to what was actually measured in the field. However, significant differences were found at many of the 146 test locations. Table 2-4 shows that the 2-hour averaged infiltration rates and their COVs in each of the four major categories were about 0.5 to 2. Although these COV values are generally high, they are much less than if compaction was ignored. These data are being fitted to conventional infiltration models, but the high variations within each of the four main categories makes it difficult to identify legitimate patterns, implying that average infiltration rates within each event may be most suitable for predictive purposes. The remaining uncertainty can be considered using Monte Carlo components in runoff models. More detailed analyses of these data will be presented in the Toronto stormwater modeling conference next year.

Table 2-4. Infiltration Rates for Different Soil Texture, Moisture, and Compaction Conditions

	Number of tests	Average infiltration rate (in/hr)	COV
noncompact sandy soils	29	17	0.43
compact sandy soils	39	2.7	1.8
noncompact and dry clayey soils	18	8.8	1.1
all other clayey soils	60	0.69	2.1

Very large errors in soil infiltration rates can easily be made if published soil maps and typical models are used for typically disturbed urban soils. Knowledge of compaction (which can be mapped using a cone pentrometer, or estimated based on expected activity on grassed areas) can be used to much more accurately predict stormwater runoff quantity.

Basic Characteristics of the Small Storm Hydrology Model

Figure 2-29 earlier showed the small storm hydrology model which describes the shape of the relationship between rainfall and runoff. Both small-scale and large-scale tests, described by Pitt (1987), obtained data to calibrate and verify this model for homogeneous impervious and pervious areas. The runoff response curve shown on Figure 2-29 departs from the x-axis at the rainfall depth when runoff begins (r_0). This depth lag corresponds to initial runoff losses. 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 dirt and debris become saturated. Between these two rain depths, infiltration losses occur.

Figure 2-29. Example pavement test runoff-rainfall plot for high intensity rains, clean and rough streets (Pitt 1987).

Figure 2-30a. Pavement infiltration rates for time since start of rain (Pitt 1987).

Figure 2-30b. Pavement infiltration rates for rain depth since start of rain (Pitt 1987).

Figure 2-31. 3-D plots showing interactions affecting infiltration rates in sandy soils.

Figure 2-32. 3-D plots showing interactions affecting infiltration rates in clayey soils.

Both small-scale and large-scale tests, described by Pitt (1987), obtained data to calibrate and verify a model for homogeneous impervious and pervious areas. The runoff response curve departs from the x-axis at the rainfall depth when runoff begins. This depth lag corresponds to initial runoff losses (detention storage, evaporation losses due to pavement cooling, and dirt and debris absorbing moisture for pavements). After some rain depth, infiltration into the ground (or pavement or through cracks) slows practically to nothing, and each additional increment of rainfall results in a similar increment of runoff. Between these two rain depths, infiltration losses occur. Figure 2-33 shows the model describing these infiltration losses. This figure plots cumulative variable runoff losses (F, inches or mm), ignoring the initial losses, versus cumulative rain (P, inches or mm), after runoff begins. The slope of this line is the instantaneous variable runoff loss (infiltration) occurring at a specific rain depth after runoff starts. A simple nonlinear model can be used to describe this relationship which is similar to many other infiltration models. 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 small storm hydrology nonlinear model for this variable runoff loss (F) is therefore:

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

Three basic model parameters were used to define the model behavior, in addition to initial runoff losses and rain depth: “a”, the intercept of the equilibrium loss line on the cumulative variable loss axis; “b”, the rate of the variable losses after equilibrium; and “g”, an exponential coefficient. If variable losses are zero at equilibrium, then “b” would be zero. Because this plot does not consider initial runoff losses, the variable loss line must pass through the origin. This model reduces to the SCS model when the “b” value is zero and “a” is S’, and when Ia is 0.16 (80% of 0.2) of “a”. This general model also reduces to the Horton equation when cumulative rain depth since the start of the event is used instead of just time since the start of rain.

Observed runoff data from both small- and large-scale tests were fitted to this equation to determine the values for a, b, and g for observed i and t (or P), and F values. In addition, outfall runoff observations from many different heterogeneous land uses were used to verify the calibrated model (Pitt 1987).

Comparison of the Small Storm Hydrology Model with the Horton Infiltration Equation

The Horton equation is used in many urban runoff models to predict infiltration losses (Skaggs, *et al*. 1969). The small storm hydrology model can be directly compared to the Horton infiltration equation. The total storm infiltration rate is:

where F(t) is an instantaneous infiltration rate. The instantaneous infiltration rate is then:

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

From the small storm hydrology model:

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

Therefore, 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 small storm hydrology model and the Horton equation are equivalent if the following relationships are simultaneously true:

Figure 2-33. Small storm rainfall-runoff infiltration model (ignoring initial abstractions) (Pitt 1987).

$bi = Fc$, or $b = Fc/i$

$-git = -kt$, or $g = k/i$

$agi = Fo - Fc$, or $a = (Fo - Fc)/gi$, or $a = (Fo - Fc)/k$.

Rearranging gives:

$Fc = ib$ (if Fc is zero, then b is also zero),

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

$k = ig$.

Based on these relationships, 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 (i) is a factor.

During the small-scale pavement runoff tests (Pitt 1987), 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. In urban hydrology studies, infiltration losses in pervious areas 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. 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 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.

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, infiltration in pavement "systems" includes infiltration through the pavement itself, infiltration through pavement cracks and seams, 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.

Comparison of the Small Storm Hydrology Model with the NRCS Curve Number Procedure

The Natural Resources Conservation Service curve number procedure (SCS 1986) is commonly used in the design of storm drainage systems. The following paragraphs illustrate how the small storm hydrology model can interface with models using curve numbers. The small storm hydrology model can be used to select curve numbers, allowing the better incorporation of the mutual drainage and flood control benefits of many water quality control measures into the design of storm drainage systems (Pitt 1987).

The NRCS CN procedure can also be compared with the small storm hydrology model and the Horton infiltration equation. The small storm hydrology model can be rewritten, knowing that $P = it$ so that $F = bP + a(1 - e^{-gP})$. However, the NRCS 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. Therefore, the NRCS S' value (maximum variable loss, without Ia, the initial abstractions) 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 NRCS) and CN in the NRCS procedure. Therefore, each curve number has a unique S' value. Because the NRCS CN procedure assumes zero final infiltration, the small storm hydrology model b value is zero and the "a" value is equal to S', as shown above. The small storm hydrology model g value was determined using a nonlinear computer program (the NONLIN module of SYSTAT - The System for Statistics, Version 3, 1986, from SYSTAT, Inc., Evanston, Ill.) for the specific F verses P relationships unique for each curve number (and S' value). The maximum runoff loss, S', which ignores initial abstractions, occurs after little rain for large curve numbers, but is not reached even after 90 mm of rain for curve numbers less than about 80.

Table 2-5 shows the fitted small storm hydrology model equation parameter g values for several curve number values, using SYSTAT's NONLIN module. This table also shows the NRCS S' values and the Horton initial infiltration rate (Fo) and decay coefficients (k) for these curve numbers. According to the small storm hydrology model, the Horton equation parameters are all related to rain intensity for impervious surfaces, and the small storm hydrology model g parameter is directly related to the curve number (Pitt 1987).

Table 2-5. Small Storm Hydrology Model and Horton Infiltration Equation Parameters for Different NRCS Curve Number Values (Pitt 1987)

Volumetric Runoff Coefficients can be Calculated for Different Surfaces and Rains using the Small Storm Hydrology Model

Table 2-6 is a summary of the volumetric runoff coefficients (Rv, the ratio of runoff to rainfall volume) for different urban surfaces and rain depths from detailed source area runoff tests and through calibrating the small storm hydrology model (Pitt 1987). Flat roofs and unpaved parking areas behave strangely similar because of similar detention storage volumes and no infiltration. Large impervious areas have the largest runoff yields because of very poor pavement under-drainage. The drainage path through the pavement base is relatively thin and very long, making it very difficult for infiltrated water to drain from the base. Street widths are much narrower than the widths of large impervious areas and the base water can drain much more effectively. Pitched roofs have no infiltration rates, but do experience limited initial losses associated with flash evaporation and sorption of moisture in leaves and other roof or gutter debris. After three inches (no longer a "small" rain) the runoff yields from all impervious surfaces are similar (within 10%), but the differences can be very large for the small rains of most concern in water quality evaluations.

Table 2-6. Summary of Volumetric Runoff Coefficients for Urban Runoff Flow Calculations (Pitt 1987).

Runoff Coefficients for Directly Connected Areas:

Rain Depth mm	Rain Depth inches	Flat roofs* (or large unpaved parking areas)	Pitched roofs*	Large impervious areas*	Small impervious areas and streets	Sandy soils	Typical urban soils	Clayey soils
1	0.04	0.00	0.25	0.93	0.26	0.00	0.00	0.00
3	0.12	0.30	0.75	0.96	0.49	0.00	0.00	0.00
5	0.20	0.54	0.85	0.97	0.55	0.00	0.05	0.10
10	0.39	0.72	0.93	0.97	0.60	0.01	0.08	0.15
15	0.59	0.79	0.95	0.97	0.64	0.02	0.10	0.19
20	0.79	0.83	0.96	0.97	0.67	0.02	0.11	0.20
30	1.2	0.86	0.98	0.98	0.73	0.03	0.13	0.22
50	2.0	0.90	0.99	0.99	0.84	0.07	0.16	0.26
80	3.2	0.94	0.99	0.99	0.90	0.15	0.24	0.33
125	4.9	0.96	0.99	0.99	0.93	0.25	0.35	0.45

*If these "impervious" areas drain for a significant length across sandy soils, the sandy soil runoff coefficients will usually be applied to these areas, however, if these areas drain across typical, or clayey soils, the runoff coefficients will be reduced, depending on the land use and rain depth, according to the following table:

Reduction factors for different rain depths (mm):

	1	3	5	10	15	20	30	50	80	125
Strip commercial and shopping centers:	0.00	0.00	0.47	0.90	0.99	0.99	0.99	0.99	0.99	0.99

Other medium to high density land uses, with alleys:	0.00	0.08	0.11	0.16	0.20	0.29	0.46	0.81	0.99	0.99
Other medium to high density land uses, without alleys:	0.00	0.00	0.11	0.16	0.20	0.21	0.22	0.27	0.34	0.46

If low density land uses, use typical or clayey soil runoff coefficients.

The impervious and roof area values are for directly connected surfaces. If runoff is allowed to drain across grass areas, then the runoff yield may significantly decrease. However, sufficient length of drainage across the pervious surface in good condition is needed. For a relatively small paved surface, short pervious drainage paths are all that are needed. If the paved area is large, or if the pervious area has clayey or compacted soils, then much longer drainage paths are needed before significant infiltration occurs.

Table 2-6 does not accurately incorporate the effects of disturbed urban soils presented earlier, but the runoff coefficients shown generally bracket the range of likely conditions expected. Some users have had good success using an intermediate soil Rv value, half way between the clayey and sandy soil conditions shown, and only using the extreme values for more unusual cases. The four urban soil categories identified earlier better represent the conditions encountered, and appropriate coefficients are currently being developed.

The runoff coefficients and indirect connection correction values were determined from calibrating the small storm hydrology model for large urban watersheds having variable complexities in Toronto and in Milwaukee (Pitt 1987). The first calibrations were conducted for simple areas. The first area was the large parking area of a commercial shopping area. The runoff coefficients for this area were used to determine the runoff relationships from large flat roofs from another shopping area that was made of mostly paved large parking and roof areas in order to determine runoff characteristics for flat roofs. The next step was to evaluate runoff data for two high density residential areas that had very little pervious areas and had all of the impervious areas directly connected. The street runoff was subtracted from the total area runoff observations to obtain information solely for pitched roofs. Finally, two medium density residential areas were studied in areas that had clayey soils and all of the impervious areas were directly connected. Roof, street and other impervious area runoff information was subtracted to obtain clayey soil runoff coefficients. Similarly, a medium density residential area was studied in an area having sandy soils to obtain sandy soil runoff coefficients. Finally, two medium density residential areas having unconnected impervious areas were studied to obtain correction coefficients.

Excellent Verification of Small Storm Hydrology Model for Many Conditions

The final runoff coefficients were verified using additional runoff data from these same areas (that were not used in the calibration efforts) and from areas located elsewhere. Figures 2-34 through 2-37 show how well the small storm hydrology model works over a wide range of rain depths and for two very different land uses. The “Post Office” site was a commercial shopping center, the “Burbank” site was a medium density residential area. These sites were monitored as part of the EPA’s NURP project in Milwaukee (Bannerman, *et al.* 1983). Figures 2-36 and 2-37 are for two residential sites monitored by the WI DNR in Superior, WI, and in Marquette, MI, during 1993 and 1994. These last two sites were compared to the small storm hydrology component of WinSLAMM with no local calibration, demonstrating the excellent fit of observed and predicted flows.

The model was subsequently calibrated for these two sites to enable better fits for the larger events. It was originally expected that this model would not work very well for very large storms, especially in areas having appreciable pervious areas, where rain intensity was expected to have a more significant effect on infiltration than for small rains. The largest rains observed for the two Milwaukee sites were greater than three inches, a very large rain that would not be expected to commonly occur. Even these rains had runoff quantities that were well predicted by this runoff model.

Example Application using the Small Storm Hydrology Model

The small storm hydrology model can be used to predict runoff volume yields for many different land uses and development conditions. It was specifically developed to determine runoff yields and corresponding water pollutant yields for small storms for stormwater quality investigations. As shown during the verification process, it is also useful for predicting runoff yields for moderate storms that are used for drainage design. If used in conjunction with a model that can account for water losses associated with stormwater controls (such as WinSLAMM, the Source Loading and Management Model, Pitt 1986 and 1992) it can also be used to show the mutual drainage benefits associated with these controls. As an example, the use of roadside swales, disconnections of impervious areas from the drainage system, or using infiltration devices, can all have dramatic benefits in reducing runoff volumes, even for relatively large rains.

The small storm hydrology model can be used to predict runoff yields associated with different land uses and development practices. It can also be used to predict sources of water within the drainage area. If the variable quality of runoff from each source area is known, then runoff pollutant yield estimates (and reductions) can also be made. WinSLAMM uses this approach. This information is very important when determining the best management strategy for water volume and runoff pollutant reduction. This example problem shows how the runoff yield predictions and sources of water for a simple area can be predicted for different rain depths. The benefits of source area disconnections are also shown.

Predicting Runoff Yields from Different Source Areas

• Calculate runoff quantity (inches) and distributions (%) by source area for the following conditions:

- Rain depths: 0.12; 0.79; 3.2 inches
- Medium density residential area (conventional curb and gutters, all impervious areas are directly connected to the drainage system and clayey soils are common), having the following surface area distribution:

pitched roofs	6%
driveways	5
sidewalks	3
streets	12
front yards	45
back yards	29

• Calculations:

		0.12 inch (3 mm) rain			0.79 inch (20 mm) rain		
area:	%	Rv	weighted Rv	contrib-ution	Rv	weighted Rv	contrib-ution
roofs	6	0.75	0.045	31 %	0.96	0.058	17 %
driveways	5	0.49	0.025	17	0.67	0.034	10
sidewalks	3	0.49	0.015	10	0.67	0.020	6
streets	12	0.49	0.059	41	0.67	0.080	24
frontyards	45	0.00	0.00	0	0.20	0.090	24
backyards	29	0.00	0.00	0	0.20	0.058	17
Total:	100	n/a	0.014	100	n/a	0.34	100

The Rv values are from Table 2-6 for the appropriate rain depths and source area. Weighted Rv values are determined by multiplying the Rv values by the percentage of the area represented. The weighted Rv values are summed to obtain a Rv value for the whole land use area. The percentage runoff yields are the ratios of the individual weighted Rv values to the summed whole area Rv.

- runoff for the 0.12 inch rain: (0.014)(0.12in)=0.017 in runoff
- runoff for the 0.79 inch rain: (0.34)(0.79in) = 0.27 in runoff
- similar calculations for the 3.2 inch rain results in a Rv of 0.48, therefore, the runoff for this rain: (0.48)(3.2 in) = 1.6 in runoff.

Figure 2-34. Verification of WinSLAMM hydrology component – Post Office commercial site, Milwaukee, WI.

Figure 2-35. Verification of WinSLAMM hydrology component – Burbank residential site, Milwaukee, WI.

Figure 2-36. Verification of WinSLAMM hydrology component – Superior, WI, test site.

Figure 2-37. Verification of WinSLAMM hydrology component – Marquette, MI, test site.

As the rain depth changes, the percentage contributions from each area also changes. For the smallest rain, all of the runoff is contributed from the directly connected impervious areas. However, pervious areas contribute almost half (44%) of the runoff for the 0.79 inch rain.

Benefits of source area drainage disconnections can also be predicted for this example. The following calculations show the effects of disconnecting all of the roof, driveway and sidewalk areas for this land use:

Original weighted Rv values:

	0.12” rain	0.79” rain	3.2” rain
roofs+ driveways+ walks	0.084	0.11	0.13
streets	0.059	0.08	0.11
yards	0	0.15	0.24
total Rv:	0.14	0.34	0.48
total runoff:	0.017”	0.27”	1.6”

With disconnections:

	0.12” rain	0.79” rain	3.2” rain
roofs+ driveways+ walks	(0)(0.084)= 0	(0.21)(0.11)= 0.023	(0.34)(0.13)= 0.044
streets	0.059	0.08	0.11
yards	0	0.15	0.24
total Rv:	0.06	0.25	0.39
total runoff:	0.01”	0.20”	1.3”
approx. % reduction:	60	25	20

The runoff contributions from the disconnected areas are decreased by the factors shown on Table 2-6 for medium density areas (with no alleys) having clayey soils. These disconnections can have significant effects on the runoff quantities generated for small rains. The runoff reductions for the larger rain will also likely be important for drainage design. Similar percentage reductions in peak runoff rates are also expected for these conditions.

Conclusions

Runoff volume is the most important hydraulic parameter needed for most water quality studies, while peak flow rate and time of concentration are the most important parameters for most flooding and drainage studies. Common small rains account for much more of the annual runoff volume than rare flooding events. Pitt (1987) showed that estimates of runoff volume could be made with only rain depth information. Other rain characteristics (including antecedent conditions, durations, intensities, etc.) did not substantially improve runoff volume predictions, but are likely needed for peak flow rate predictions.

The literature indicates that both initial runoff abstractions (mostly detention/storage) and continuous runoff losses (infiltration) are important for impervious surfaces. Recent work with disturbed urban soils has also shown that care must be taken when using soil maps for developed conditions. The small storm hydrology model successfully predicts runoff from several types of paved, roofed, and disturbed soil urban surfaces. This model was shown to accurately predict runoff volumes for a wide range of rain conditions.

This model was used to examine long-term rain conditions at many locations throughout the U.S. to indicate the significance of small and moderate sized rains in stormwater management. These smaller rains, compared to the typical “design storm” rains used for drainage system design, contribute the vast majority of stormwater pollutants. Stormwater control practices must therefore effectively address these smaller storms to provide effective pollutant and flow reduction schemes.

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