28 Urban Hydrology

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28.1 Introduction

The transformation of precipitation into surface runoff is depicted in Figures 28.1 and 28.2. Figure 28.1 illustrates the relationship between the rainfall process, infiltration to the subsurface, generation of overland flow, and flow into the man-made drainage system. Among the many variables that describe these processes mathematically are the width, area, percent imperviousness, ground slope, roughness parameters of the land cover for both impervious and pervious fractions, and several infiltration rate parameters that depend upon methods chosen. Eventually, the runoff makes its way into inlets of the man-made drainage system. Figure 28.2 illustrates the transport of runoff through both natural and man-made conduits, as well as routing through a detention structure. Whether natural or man-made, the length, geometry, slope, and roughness characteristics must be specified for each conduit. Natural channels can be approximated with triangular, trapezoidal, and other cross-sectional shapes. For storage elements such as detention basins (there is also storage along a channel), stage–discharge curves and other geometric data are required. Any mathematical model is an abstraction of the actual physical system it attempts to simulate, and uncertainty in the values of many model parameters requires field measurements and calibration.

![Figure 28.1](image_url) Rainfall, infiltration, overland flow, and flow into drainage system.

Subcatchment data: Width, area, percent imperviousness, ground slope, Manning’s n for impervious and pervious areas, and infiltration rate parameters.
28.2 Statistical Analysis of a Rainfall Time Series

It has been recognized for over three decades that the large number of factors that affect surface runoff quantity (rainfall duration, intensity, time between events, volume, infiltration, antecedent soil moisture, etc.) and surface runoff quality (pollutant loads and buildup between storms, surface washoff, transport, kinetic interactions, etc.) prevents the exclusive use of any single event for proper analysis and design. Regardless of whether water quantity or quality is the primary objective, long-term historical rainfall data are required. An hourly or shorter-interval precipitation time series at least 30–40 years long is desirable. The purpose in quantitative analysis of the rainfall time series is to summarize the variables of interest (depth, duration, intensity, and time between storm events) and to statistically characterize the rainfall record to assess the probability of occurrence of storm events of various magnitudes.

To properly analyze the rainfall time series, storm events must first be defined in terms of their statistical independence (see Figure 28.3). A common approach is to derive a minimum interevent time (MIT), such that the intervals between storm event midpoints are nearly exponentially distributed [12].
Trial values of the MIT are chosen until a coefficient of variation (COV) near 1.0 is finally obtained for the time interval between event midpoints.

A reliable source of data is the National Oceanic and Atmospheric Administration (NOAA) National Climatic Data Center (NCDC): access to a geographically comprehensive number of rainfall stations and their data is available online. The historical record of hourly precipitation from 1948 to 2011 at Raleigh-Durham (RDU) Airport (located only 8 miles from Durham, NC) was analyzed to obtain storm event statistics, summarized in Table 28.1. This represents over 6300 storms using a MIT of 6 h of dry weather to separate independent storm events. The average storm duration was 6.9 h.

The annual variability in the rainfall record is presented in Figure 28.4, with a mean annual rainfall of 1057 mm. Figure 28.5 shows that the mean monthly rainfall is distributed rather uniformly throughout the year, with hurricane-related extremes occurring from June to October. An analysis of the frequency distribution of storm depths reveals a large number of storms between total rainfall depths 0.254 and 4.06 mm, with almost 90% of all storms in the record less than about 25.4 mm. Capturing the “first flush”

<table>
<thead>
<tr>
<th>Storm Event Variable</th>
<th>Mean</th>
<th>COV</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. of storms* per year</td>
<td>99.523</td>
<td>0.11</td>
</tr>
<tr>
<td>Depth (mm)</td>
<td>10.668</td>
<td>0.425</td>
</tr>
<tr>
<td>Intensity (mm/h)</td>
<td>1.778</td>
<td>1.340</td>
</tr>
<tr>
<td>Duration (h)</td>
<td>6.873</td>
<td>1.097</td>
</tr>
<tr>
<td>Time between events (h)</td>
<td>87.372</td>
<td>1.002</td>
</tr>
<tr>
<td>Total no. of storms*</td>
<td>6309</td>
<td></td>
</tr>
</tbody>
</table>

*Note: Number of storms is based on a MIT of 6 h, which yielded a COV very near 1.0 for time between events for the rainfall time series. Interevent times have an exponential distribution (for which the mean equals the standard deviation, ∴ COV = 1.0).
would control 90% of all storm events. Figure 28.6 shows clearly that short-duration, high-intensity storms occur during the summer months. Figure 28.7 shows that storms are more frequent during the summer months, less frequent from September to December: there are also water quality implications during large intervals between storms, in terms of pollutant load accumulations on the ground. The rainfall time series statistics in Table 28.1 are an integral part of a code used to predict the expected pollutant removal performance of detention facilities based on a probabilistic model, discussed later.

28.3 Overland Flow

Overland flow is very thin sheet flow that develops after precipitation falls over a sloped land surface and infiltration occurs into the subsurface. The depth and flow rate of this thin sheet of water depends, among others, on the rainfall intensity, ground surface characteristics (e.g., roughness and slope), antecedent moisture, and immediate subsurface conditions (e.g., hydraulic conductivity). By neglecting the acceleration and pressure terms in the momentum equation for dynamic waves, the kinematic-wave
model [7] substitutes a steady uniform flow (stage–discharge) relationship for the momentum equation. However, unsteady flow is preserved through the continuity equation.

The dynamic wave equations for overland flow consist of continuity and momentum equations, as follows, respectively [4]:

\[ \frac{\partial y}{\partial t} + V \frac{\partial y}{\partial x} + y \frac{\partial V}{\partial x} = i - f \] (28.1)

\[ \frac{\partial V}{\partial t} + V \frac{\partial V}{\partial x} + g \frac{\partial y}{\partial x} = g \left( S_f - S_b \right) + \left[ (i - f) \frac{V}{y} \right] \] (28.2)

where
- \( y \) is depth of water (L)
- \( S_f \) is friction slope (L/L)
- \( S_b \) is bed slope (L/L)
- \( V \) is water velocity (L/T)
- \( i \) is rainfall intensity (L/T)
- \( f \) is infiltration rate (L/T)
- \( g \) is acceleration of gravity (L/T²)
- \( t \) is time (T)
- \( x \) is distance (L)

These equations are for moderately wide overland flow and small bottom slope. An order-of-magnitude analysis of Equation 28.2 shows that \((i - f)\) has a negligible effect on the flow dynamics [4]. The flow per unit width \( q \) (L²/T) is equal to \( Vy \). Thus, Equation 28.1 may be written as

\[ \frac{\partial q}{\partial x} + \frac{\partial y}{\partial t} = i - f \] (28.3)

The simplest distributed routing method is the kinematic-wave model, which neglects the local acceleration, convective acceleration, and pressure terms in the momentum equation (thus, friction and gravity

**FIGURE 28.7** Time between independent storm events.
forces essentially balance each other, \( S_f = S_0 \). The most common formulas relating water velocity, slope, and catchment roughness are [10]:

Manning formula

\[
V = \frac{k}{n} R^{2/3} \sqrt{S_0}
\]  

(28.4)

Chezy formula

\[
V = C \sqrt{R S_0}
\]  

(28.5)

where

- \( V \) is water velocity (L/T)
- \( n \) is the Manning friction coefficient (dimensionless)
- \( R \) is the hydraulic radius (L)
- \( C \) is the Chezy coefficient (L^{1/2}/T)
- \( k = 1.486 \) for English system units and 1.0 for the SI system
- the bed slope \( S_0 \) replaces the friction slope \( S_f \)

For wide rectangular sections (e.g., overland flow), the hydraulic radius reduces to water depth \( y \), and the momentum equation for kinematic waves reduces to

\[
q = \alpha y^m
\]  

(28.6)

where, for **fully turbulent flow**, using Manning

\[
\alpha = \frac{k}{n} \sqrt{S_0} \quad m = \frac{5}{3}
\]  

(28.7)

using Chezy

\[
\alpha = C \sqrt{S_0} \quad m = \frac{3}{2}
\]  

(28.8)

Values of exponent \( m \) for laminar and mixed laminar-turbulent conditions are tabulated by Ponce [10]. For overland flow problems using the kinematic wave, the continuity and momentum equations can be combined into one differential equation, as follows:

\[
\begin{aligned}
\frac{\partial y}{\partial t} + \frac{\partial q}{\partial x} &= i - f \\
\Rightarrow \frac{\partial y}{\partial t} + \alpha \frac{\partial (y^m)}{\partial x} &= i - f
\end{aligned}
\]  

(28.9)

### 28.4 Channel Flow

The distributed routing models allow us to compute flow rate and water level variation through space and time: a major advantage over the lumped model in terms of the design criteria of any storage structure, such as a detention pond or reservoir. Similar to the overland flow problems, by eliminating some terms in the momentum equation of the Saint Venant equations, alternative distributed flow routing models are obtained. The dynamic wave equations for channel flow consist of continuity and momentum equations, as follows, respectively [1]:

\[
\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = q(x,t)
\]  

(28.10)
\[
\frac{1}{A} \frac{\partial Q}{\partial t} + \frac{1}{A} \frac{\partial}{\partial x} \left( \frac{Q^2}{A} \right) + g \frac{\partial y}{\partial x} - g \left( S_b - S_f \right) = 0 \quad (28.11)
\]

where
- \( y \) is depth of water (L)
- \( S_f \) is friction slope (L/L)
- \( S_b \) is bed slope (L/L)
- \( Q \) is flow rate (L^3/T)
- \( q(x, t) \) is the net lateral inflow per unit length of channel (L^2/T)
- \( g \) is acceleration of gravity (L/T^2)
- \( A \) is the channel cross-sectional area (L^2), \( t \) is time (T)
- \( x \) is distance (L)

In Equation 28.11, the first, second, and third terms represent local acceleration, convective acceleration, and pressure force, respectively. The bed slope and friction slope in the last term represent the gravity force and friction force.

Again, the simplest distributed routing method is also the \textit{kinematic-wave} model, which neglects the local acceleration, convective acceleration, and pressure terms in the momentum equation for dynamic waves. For open channel flows, the continuity and momentum equation and their combined form for kinematic waves are given as follows:

\[
\begin{align*}
\frac{\partial A}{\partial t} + \frac{\partial Q(x, t)}{\partial x} &= q(x, t) \\
Q &= \alpha A^m
\end{align*}
\quad (28.12)
\]

where, for \textit{fully turbulent} flow,

Manning

\[
\alpha = \frac{k \sqrt{S_b}}{n \ P^{2/3}} \quad m = \frac{5}{3}
\]

Chezy

\[
\alpha = C \sqrt{\frac{S_b}{P}} \quad m = \frac{3}{2}
\quad (28.13)
\]

Here
- \( A \) is the channel cross-sectional area (L^2)
- \( Q \) is the flow rate (L^3/T)
- \( n \) is the Manning friction coefficient (dimensionless)
- \( C \) is the Chezy coefficient (L^{1/2}/T)
- \( P \) is the wetted perimeter (L)
- \( S_b \) is the bed slope (L/L)
- \( t \) is time (T)
- \( q(x, t) \) is the net lateral inflow per unit length of channel
- \( x \) is the distance along the flow axis (L).

As before, \( k = 1.486 \) for English system units and 1.0 for the SI system.
28.5 Infiltration

Most urban simulation models have used the Horton equation for prediction of infiltration capacity into the soil as a function of time,

\[ f_{cap} = f_\infty + (f_0 - f_\infty) e^{-\alpha t} \]  \hspace{1cm} (28.14)

where
- \( f_{cap} \) is the infiltration capacity into soil, say in./h
- \( f_\infty \) is the minimum or ultimate value of \( f \) (at \( t = \infty \)), in./h
- \( f_0 \) is the maximum or initial value of \( f \) (at \( t = 0 \)), in./h
- \( t \) is the time from beginning of storm, sec
- \( \alpha \) is the decay coefficient, 1/time

The actual infiltration is given by

\[ f(t) = \min[f_{cap}(t), i(t)] \]  \hspace{1cm} (28.15)

where
- \( f \) is the actual infiltration into the soil (say in./h)
- \( i \) is the rainfall intensity (say in./h)

Equation 28.15 simply states that the actual infiltration will be the lesser of actual rainfall and available infiltration capacity.

Typical values for parameters \( f_0 \) and \( f_\infty \) are often greater than typical rainfall intensities. Thus, when Equation 28.14 is used such that \( f_{cap} \) is a function of time only, then \( f_{cap} \) will decrease even if rainfall intensities are very light. This results in a reduction in infiltration capacity regardless of the actual amount of entry of water into the soil. Thus, to correct this problem, the integrated form of the Horton equation may be used

\[ F(t_p) = \int_0^{t_p} f_{cap} \, dt = f_\infty t_p + \frac{f_0 - f_\infty}{\alpha} \left(1 - e^{-\alpha t_p}\right) \]  \hspace{1cm} (28.16)

where \( F \) is the cumulative infiltration at time \( t_p \) (inches or mm). The true cumulative infiltration will be

\[ F(t) = \int_0^t f(\tau) \, d\tau \]  \hspace{1cm} (28.17)

where \( f \) is given by Equation 28.15. Equations 28.16 and 28.17 may be used to define the equivalent time, \( t_p \). That is, the actual cumulative infiltration given by Equation 28.17 is equated to the area under the Horton curve (given by Equation 28.16), and the resulting equation is solved for \( t_p \) and serves as its definition. Unfortunately, the following equation

\[ F = f_\infty t_p + \frac{f_0 - f_\infty}{\alpha} \left(1 - e^{-\alpha t_p}\right) \]  \hspace{1cm} (28.18)

cannot be solved explicitly for \( t_p \): It must be done iteratively. Note that \( t_p \leq t \) that states that the time \( t_p \) on the cumulative Horton curve will be less than or equal to the actual elapsed time. This also implies
that available infiltration capacity \( f_{\text{cap}}(t_p) \) will be greater than or equal to that given by Equation 28.14. Thus, \( f_{\text{cap}} \) will be a function of the actual water infiltrated and not a function of time only, with no other effects. The original Horton model does poorly when light rainfall falls early, since it decays in time independently of accumulated infiltration. The integrated form of the Horton equation is applicable to urban environments, particularly for relatively short storm events. It should be noted that the Horton equation is an approximate solution to the Richards equation: 1D, unsteady, unsaturated flow—under several simplifying assumptions.

An alternative model is the Green–Ampt formulation, better suited for nonurban environments. Green and Ampt [5] derived the first physically based equation describing the infiltration of water into a soil. It has been refined and adapted for use in computer codes. A schematic representation of the Green–Ampt model is presented in Figure 28.8. In Figure 28.8, the wetting front has penetrated to a depth \( L \) in time \( t \) since infiltration began. Water is ponded to a small depth \( h_0 \) on the soil surface. The Green–Ampt equations for cumulative infiltration \( F \) and infiltration rate \( f \) are given by

\[
F(t) - \psi \Delta \theta \ln \left[ 1 + \frac{F(t)}{\psi \Delta \theta} \right] = Kt
\]  
\[
f(t) = K \left[ \frac{\psi \Delta \theta}{F(t) + 1} \right]
\]  

The parameters are defined as follows:
- \( \psi \) is the wetting front soil suction head (cm)
- \( \theta \) is the moisture content
- \( K \) is the hydraulic conductivity (cm/h)

Equation 28.19 is commonly solved by Newton–Raphson iteration. Once \( F \) is found, then the infiltration rate is obtained. The parameters are well documented for a variety of soil classes (sand, sandy loam, loam, silt loam, sandy clay loam, clay loam, silty clay loam, sandy clay, and clay).
28.6 Detention Storage

The basic governing differential equation, the continuity equation, is given by

\[ \frac{dV}{dt} = Q_{in}(t) - Q_{out}(h) \]  

(28.21)

The change in volume \(dV\) due to a change in depth \(dh\) may be expressed as

\[ dV = A(h) \, dh \]  

(28.22)

such that the continuity equation can now be expressed as

\[ \frac{dh}{dt} = \frac{Q_{in}(t) - Q_{out}(h)}{A(h)} \]  

(28.23)

A common method to numerically integrate ordinary differential equations is the fourth-order Runge–Kutta method, such that the head is approximated as follows:

\[ h_{n+1} = h_n + \frac{1}{6} \Delta t \left[ k_1 + 2k_2 + 2k_3 + k_4 \right] \]  

(28.24)

where

\[ k_1 = f\left(h_n, t_n\right) \]
\[ k_2 = f\left(h_n + \frac{1}{2} k_1 \Delta t, t_n + \frac{\Delta t}{2}\right) \]
\[ k_3 = f\left(h_n + \frac{1}{2} k_2 \Delta t, t_n + \frac{\Delta t}{2}\right) \]
\[ k_4 = f\left(h_n + k_3 \Delta t, t_n + \Delta t\right) \]

More sophisticated urban hydrologic simulation programs, such as the well-known U.S. EPA Storm Water Management Model (SWMM5), use a fifth-order accurate Runge–Kutta method with adaptive step size control. This means that such a code adapts the step size to achieve some predetermined accuracy with minimum computational effort [11].

28.7 Surface Water Quality

In surface water quality, pollutant accumulation and washoff processes are mathematically approximated. The assumptions of these processes are as follows: (1) the amount of pollutant, which can be removed during a storm event, depends on rainfall duration and initial quantity of pollutant available; (2) chemical changes or biological degradation does not affect the pollutant decay during the runoff process; and (3) the amount of pollutants percolating into the soil by infiltration is insignificant [8]. The exponential decay of solid buildup is generally represented by

\[ p_{n,j,t} = p_{max,j} \left[ 1 - e^{-\lambda_{n,j} \Delta t} \right] \]  

(28.25)

where

\[ p_{n,j,t} \] is the loading of pollutant \(j\) on subcatchment \(n\) at time \(t\) [lb/acre (kg/ha)]
\[ p_{max,j} \] is the maximum (asymptotic) loading of pollutant \(j\) on subcatchment \(n\) [lb/acre (kg/ha)]
\[ \lambda_{n,j} \] is power exponent coefficient for pollutant \(j\) on subcatchment \(n\) [day\(^{-1}\)]
\( t \) is accumulation time [day]
The washoff is calculated by making it proportional to runoff rate at each time step of simulation as follows:

\[ P_t = \frac{dP}{dt} = -ar^bP_o \]  

(28.26)

where
- \( P_t \) is the pollutant load washed off at time \( t \) [lb/s (kg/s)]
- \( P_o \) is the load available for washoff at time \( t \) [lb (kg)]
- \( r \) is the runoff rate [in/h (mm/h)]
- \( a \) is the washoff coefficient, including conversion units
- \( b \) is a power constant

Pollutant transport in overland flow and through urban storage/treatment systems such as pipes, channels, and detention basins may be represented by the 1D version of the classical advective–dispersive equation (28.9):

\[ \frac{\partial C}{\partial t} = \frac{\partial}{\partial x} \left[ E \frac{\partial C}{\partial x} - UC \right] + \sum_{i=1}^{n} S_i \]  

(28.27)

where
- \( C \) is the concentration of pollutant [M/L^3]
- \( t \) is time [T],
- \( E \frac{\partial C}{\partial x} \) is the mass flux due to longitudinal dispersion along the flow axis (x-direction) [M/L^2]
- \( UC \) is mass flux due to advection by the fluid containing the mass of pollutant [M/L^2T]
- \( S_i \) is the sources or sinks of the substance [M/L^3T]
- \( n \) is the number of sources or sinks
- \( U \) is the flow velocity [L/T]
- \( E \) is the longitudinal dispersion coefficient [L^2/T]

The mass balance equation for a well-mixed, variable volume unit is derived from Equation 28.27 and is given by Medina et al. [9]:

\[ \frac{d(VC)}{dt} = I(t)C^i(t) - O(t)C(t) - KC(t)V(t) \]  

(28.28)

where
- \( V \) is the reservoir volume [L^3]
- \( C^i \) is the influent pollutant concentration [M/L^3]
- \( C \) is the effluent and reservoir pollutant concentration [M/L^3]
- \( I \) is the inflow rate [L^3/T]
- \( O \) is the outflow rate [L^3/T]
- \( t \) is time [T]
- \( K \) is the first-order decay coefficient [1/T]

This equation is solved numerically over time for concentration, using average values for quantities that might change over a time step, such as flow rate and volume.

28.8 Detention Pond Long-Term Average Pollutant Removal Efficiency

The basic probabilistic methodology was developed by Di Toro and refined by Di Toro and Small [2] and adapted to wet ponds (sedimentation devices) and percolation (infiltration and recharge) devices by
Driscoll et al. [3]. The long-term average performance is computed from the statistical properties of the detention basin inflows and a few design characteristics. Removal due to sedimentation under dynamic (flow-through) conditions is given by

\[ R = 1 - \left( 1 + \frac{1}{n} \left( \frac{V_s}{Q/A} \right)^n \right) \]  

where

- \( R \) is the fraction of initial solids removed (\( R \times 100 = \% \) removal)
- \( V_s \) is the settling velocity of particles
- \( Q/A \) is the rate of applied flow divided by the surface area of the basin (an “overflow velocity,” often designated overflow rate)
- \( n \) is a measure of the degree of turbulence or short circuiting which tends to reduce removal efficiency; typically, \( 1 \leq n \leq 5 \), with \( n = 3 \) for good performance and \( n > 5 \) for very good performance.

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

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

where

- \( R_L \) is the long-term average fraction removed
- \( R_M \) is the fraction removed at the mean runoff rate
- \( V_q \) is the COV of runoff flow rates
- \( r \) is the \( 1/\text{CV}^2 \) (reciprocal of square of \( CV_q \))
- \( Z \) is the maximum fraction removed at very low rates

Removal under quiescent conditions is very significant since average storm duration in the United States is 6 h and the average interval between storms is from 3 to 4 days. The volume of a detention basin relative to the volumes of runoff events routed through becomes the principal factor in pollutant removal effectiveness. For storm volumes that are gamma distributed, the fraction not captured (over all storms) may be represented by

\[ f_s = \frac{1}{G(\tau_1)G(\tau_2)} \int_{q=0}^{\infty} q^6 \exp \left[ -\tau_1 q \right] \int_{\Delta=0}^{\infty} \left[ \Delta + \frac{\forall}{q} \right]^{\tau_2-1} \exp \left[ -r_2 \Delta \right] d\Delta dq \]

where

- \( CV_q \) is the COV of runoff flow rates
- \( CV_d \) is the COV of runoff durations
- \( q \) is the storm runoff flow rate
- \( \Delta \) is the average time interval between storm midpoints
- \( \forall \) is the basin effective volume, divided by mean storm runoff volume
- \( G(\tau_1) \) is the gamma function of \( \tau_1 \)
- \( G(\tau_2) \) is the gamma function of \( \tau_2 \)
- \( f_s \) is the fraction of all volumes NOT captured by the detention basin
Detention Pond Long Term Pollutant Removal Metric/SI Units

<table>
<thead>
<tr>
<th>Rainfall Stats:</th>
<th>Mean</th>
<th>CV</th>
</tr>
</thead>
<tbody>
<tr>
<td>Volume (in or mm)</td>
<td>10.668</td>
<td>1.394</td>
</tr>
<tr>
<td>Intens (in/h or mm/h)</td>
<td>1.778</td>
<td>1.340</td>
</tr>
<tr>
<td>Duration (h)</td>
<td>6.873</td>
<td>1.097</td>
</tr>
<tr>
<td>Delta (h)</td>
<td>87.372</td>
<td>1.002</td>
</tr>
<tr>
<td>Runoff COEF (Rv)</td>
<td>0.30</td>
<td></td>
</tr>
<tr>
<td>Drainage area (acres or hectares)</td>
<td>80.94</td>
<td></td>
</tr>
<tr>
<td>Basin surf area (sq ft or sq m)</td>
<td>5774.18</td>
<td></td>
</tr>
<tr>
<td>Basin depth (ft or m)</td>
<td>2.74</td>
<td></td>
</tr>
<tr>
<td>Short circ param</td>
<td>2.50</td>
<td></td>
</tr>
</tbody>
</table>

Mean runoff rate (QR) = 431.72 cu m/h
Overflow rate (QR/A) = 0.07 m/h
Vol capac factor (VB/VR) = 6.12
Basin Surface Area / Catchment Area, in % = 0.71

Size Frac % Remove Dynamic % Remove Quiescent % Remove Combined Sett veloc M / HR Vol factor VE / VR
1 0.7 76.8 77.5 0.01 5.36
2 3.1 79.0 82.1 0.09 6.06
3 6.9 79.2 86.1 0.46 6.12
4 7.8 79.2 87.0 2.13 6.12
5 7.9 79.2 87.0 19.81 6.12
Avg 5.3 78.7 84.0

Avg total suspended solids removal = 84.0
Avg total phosphorus removal = 58.8
Avg lead removal = 75.6
Avg tkn, bod, cu, zn removal = 42.0

FIGURE 28.9 WetDet model output.

The double integral cannot be evaluated analytically. A numerical integration technique using Laguerre quadrature is applied to approximate the integral with a weighted polynomial. An application follows using the rainfall statistics presented earlier in Table 28.1, for a hypothetical West Campus detention basin. Model output is presented later in Figure 28.9. As expected, detention removes a significant percentage of total suspended solids and lead (found to a large extent in suspended solids) and is much less efficient in removing dissolved pollutants.

The results previously presented are specific to a ratio of detention pond surface area to drainage area contributing of 0.71%. This is a critical factor in long-term pollutant removal performance, as shown in Figure 28.10, when the detention pond surface area is varied to obtain a range of ratios.

28.9 U.S. Environmental Protection Agency

Storm Water Management Model

EPA SWMM5 is a dynamic rainfall/runoff simulation model that can be used for single storm event simulation or continuous simulation of multiple storms of a specific watershed. The model calculates the flow and water quality constituents of the watershed based on hydraulic parameters set by the user and provides graphs of simulation results such as flow hydrographs and water quality concentrations. The model is widely used to plan, analyze, and control storm water runoff; to design drainage system components; and to evaluate watershed management of both urban and nonurban areas [6,13]. Images of urban areas obtained from GIS mapping software can be displayed as backdrops to visualize the physical system being simulated.
The runoff component of SWMM simulates surface runoff over subcatchment areas, as generated from rainfall input. Then, SWMM routes this runoff with the transport component through a conveyance system of pipes, channels, storage/treatment devices, pumps, and regulators. SWMM calculates the quantity and quality of runoff generated within each subcatchment and the flow rate, flow depth, and quality of water in each pipe and channel during a simulation period comprised of multiple time steps. SWMM inputs include precipitation data, subcatchment delineation, pipe system characteristics, and soil properties. The watershed profile is formed by defining input data such as slope, area, and imperviousness of subcatchments; length, cross section, height, and slope of conduits; and porosity, hydraulic conductivity, and field capacity of aquifers, into the program. Outputs of the model include change of flow rate (hydrograph) and change of concentration (pollutograph) through time and daily, monthly, annual, and total simulation summaries (for continuous simulation) and statistical frequency analysis. SWMM is also capable of viewing results in tabular and graphical form.

While calculating surface runoff, precipitation, and flow from upstream subcatchments that contribute to the inflow, infiltration and evaporation contribute to the outflow. For flow routing in conduits, steady flow, and kinematic, diffusion and dynamic wave routing options may be used. In addition, for infiltration calculation, three options exist, which are the integrated Horton method, the Green–Ampt method, and the SCS curve number method. EPA SWMM simulates surface runoff quality by using exponential, power, and saturation functions for buildup and exponential, rating curve, and event mean concentration functions for washoff.

28.10 Application to Duke University Campus
Watershed in Durham, North Carolina

The Duke University West Campus watershed is located in Durham, North Carolina. Runoff from the West Campus of Duke University is simulated by using the EPA SWMM to predict surface runoff and quality and transport through the drainage system. The first formal application of EPA SWMM to the Duke Campus was completed by Mahi [8]. In order to be able to use EPA SWMM for surface runoff and

![Figure 28.10](https://example.com/figure28.10.png)

**FIGURE 28.10** Detention pond performance as a function of pond surface area.
channel flow predictions, the model needed to be calibrated against measured rainfall and runoff data. For this purpose, a rain gage was placed on the roof of Hudson Hall (Pratt School of Engineering), and several storm events were measured in 15 min intervals. About 30 storms were recorded from 1994 to 1995: these were grouped into summer and winter storms for calibration purposes. In order to measure the flow rate, an aluminum compound weir was placed at the outlet of conduit 210, near node 345, a 1.83 m (6 ft) diameter pipe. Typically, pressure transducers were located a few feet upstream of the weirs, transmitting date, time of day, and depth of flow data to electronic data loggers. Several model parameters (e.g., infiltration, depression storage, and Manning’s roughness) were adjusted to match predicted versus observed flows.

Figure 28.11 illustrates the location of the weirs used for calibration (near node 345). The rapid urbanization (Medical Center) upstream of this measurement station over the past 5 years resulted in the decision to consider the design and construction of a large detention pond to control storm water flowing off-campus (at the Erwin Road outfall) into the City of Durham. Detention ponds are considered one of the most effective runoff control devices; in particular, to reduce peak flows, settle particulate matter, while also reducing some pollutant concentrations in the outflow. They are not as effective in reducing dissolved fractions of contaminants, as shown earlier. The effect of a large detention pond in capturing a relatively small storm (storm of February 15–17, 1995, for which runoff measurements were available) is presented in Figure 28.12. Conduit 203 is upstream of the simulated detention pond. Conduit 202 is downstream from the pond and shows the pond storage effects on the hydrograph. The response of the catchment at the Erwin Road outfall (node 339) to the most intense rainfall event (May 11, 2011) ever recorded in Durham, NC, is presented in Figure 28.13.

### 28.11 Summary and Conclusions

Although the basic fluid mechanics principles of urban hydrologic processes presented in this chapter have been understood for decades by professional hydrologists, dramatic improvements in visualization techniques and ease-of-model-execution features have accelerated the learning process for model application by more inexperienced users. This trend has its drawbacks, as unfamiliarity with the governing equations (including their limitations) and black box application of these numerical models will likely result in completely unrealistic predictions. Preparation of the extensive input data required for these comprehensive models can be a tedious and time-consuming exercise. Calibration of these models with
measured rainfall, runoff, and water quality data is an essential component of a successful simulation, as described in the previous section. Validation with data sets independent of those used for calibration is desirable.

The models are best suited for development of urban planning scenarios and comparison of runoff control alternatives for a multitude of rainfall events or continuous rainfall time series.

References