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SYNTHETIC RUNOFF CAPTURE AND DELIVERY CURVES FOR STORM WATER QUALITY CONTROL DESIGNS

by

James C.Y. Guo, Professor, Department of Civil Engineering, University of Colorado at Denver, Denver, CO
Ben Urbonas, Chief, Master Planning, Urban Drainage and Flood Control District, Denver, CO 80211

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Abstract

From 1989 through 1996, Urban Drainage and Flood Control District in coordination with the University of Colorado at Denver has dedicated to the development of the concept of storm water quality runoff capture volume (QWCV). Before 1996, the major effort was to analyze tens of hundreds of individual events delimited from a continuous record. A serial of design charts and empirical formulas were published for determining the storm water capture volumes for storm water quality control designs (Urbonas, Guo, and Tucker, 1989), (Guo and Urbonas, 1996), and (Rosener, Urbonas, and Guo 1996).

Rainfall event separation process can be subjective, depending on how the minimum interevent time is chosen. In general, the longer the rainfall event separation time is, the higher the average rainfall event depth will be. To improve the consistency among the rainfall data bases, this paper presents a mathematical model by which a continuous rainfall record can be directly converted into storm water runoff capture curves. Applying the exponential distribution to a complete rainfall data series, the normalized *runoff capture curve* was derived in this study to describe the non-exceedance probability distribution of runoff depths. Similarly, the normalized *runoff delivery curve* was also developed to describe the non-exceedance probability distribution of runoff rates. These two curves provide necessary and important design information by which both the trickle channel and the WQCB can be sized on a consistent basis of overflow risk.

Key Words: Stormwater, Retention, Detention, Water Quality, Urban, Hydrology.

INTRODUCTION

In current practice, the associated overflow risk for a WQCV can be derived by continuous rainfall/runoff simulation techniques or stochastic methods using Monte Carlo simulation. For instance, Guo and Urbanas (1996) applied the point rainfall-runoff approach to construct the runoff capture curve that defines the non-exceedance probability for a selected WQCV. Guo and Hughes (2001) investigated the distribution of rainfall event-depths and derived the synthetic runoff capture curves for the design of infiltration basins. These efforts were to aim at the development of an alternative to replace the lengthy simulation process when dealing with a complete rainfall data series. Along with such an effort, this study presents an attempt to construct the normalized runoff capture curve for WQCB designs and also the normalized runoff delivery curve for trickle channel designs.

The derivation of runoff capture curves begins with the understanding of general characteristics of complete rainfall data series. A large amount of rainfall data was analyzed in this study, and it was found that the exponential distribution can reasonably describe the distribution of rainfall event-depths and be adopted to generate both synthetic runoff capture and delivery curves using the average rainfall event-depth as the normalizing parameter. The synthetic runoff capture curves generated by the mathematical model were then compared with those generated by the continuous simulation techniques using the long-term continuous rainfall record observed in seven major cities in the United States. Close agreements have been achieved. A synthetic runoff capture curve provides a consistent basis to assess the overflow risk for WQCB designs, and can be localized by the average rainfall event depth at the basin site.

FREQUENCY DISTRIBUTION FOR RAINFALL EVENT-DEPTHS

Traditionally, urban hydrology has been developed with an emphasis on extreme events, i.e. minor and major storms. The flood-frequency curve provides a basic relationship between flood magnitude and overflow risk. With a pre-selected overflow risk, the design capacity of a flood control facility can be determined. However, such a frequency-based approach developed for extreme events has been found no longer suitable for WQCV facilities. As an alternative to the traditional approach developed for *minor* and *major* design events, it was suggested that the WQCB be sized to capture the most *micro* events at the magnitude of the first flush volume (EPA in 1986, EPA in 1983). Of course, any storm water detention system can be designed to achieve the multiple purposes by addressing the micro, minor, and major storm events altogether.

In this study, the complete rainfall data series were investigated using the 20- to 30-year hourly continuous rainfall data recorded at seven metropolitan cities from various climatologic regions in the United States, including Seattle, WA, Sacramento, CA, Phoenix, AZ, Denver, CO, Cincinnati, OH, Tampa, FL, and Boston, MA. Each continuous rainfall record was divided into individual storm events by assigning a storm separation time that is defined as a minimum time period of no rain. According to EPA studies (EPA 1986), a rainfall separation time of six hours is used to divide the seven continuous rainfall records into 1500 to 2000 individual events, depending on the length of rainfall records. The distribution and statistics of rainfall event depths were then calculated for all events at each station. For example, using a rainfall separation time of 6-hours, the 30-year hourly continuous rainfall record observed at Denver, CO is divided into 1690 individual storm events. A frequency analysis is then conducted by dividing the range of the observed rainfall event-depths into 30 equal intervals. The probability of occurrence within an interval is defined as the ratio of the number of events observed to the total number of observations. The corresponding frequency is the ratio of the probability of occurrence to the width of the interval. Figures 1 and 2 present the frequency distributions of rainfall event-depths for Denver, CO, and Seattle, WA. These sets of rainfall data show that a complete rainfall data series consists of a large number of small events, and the frequency curves exhibit decay characteristics as the magnitude of rainfall depth increases.

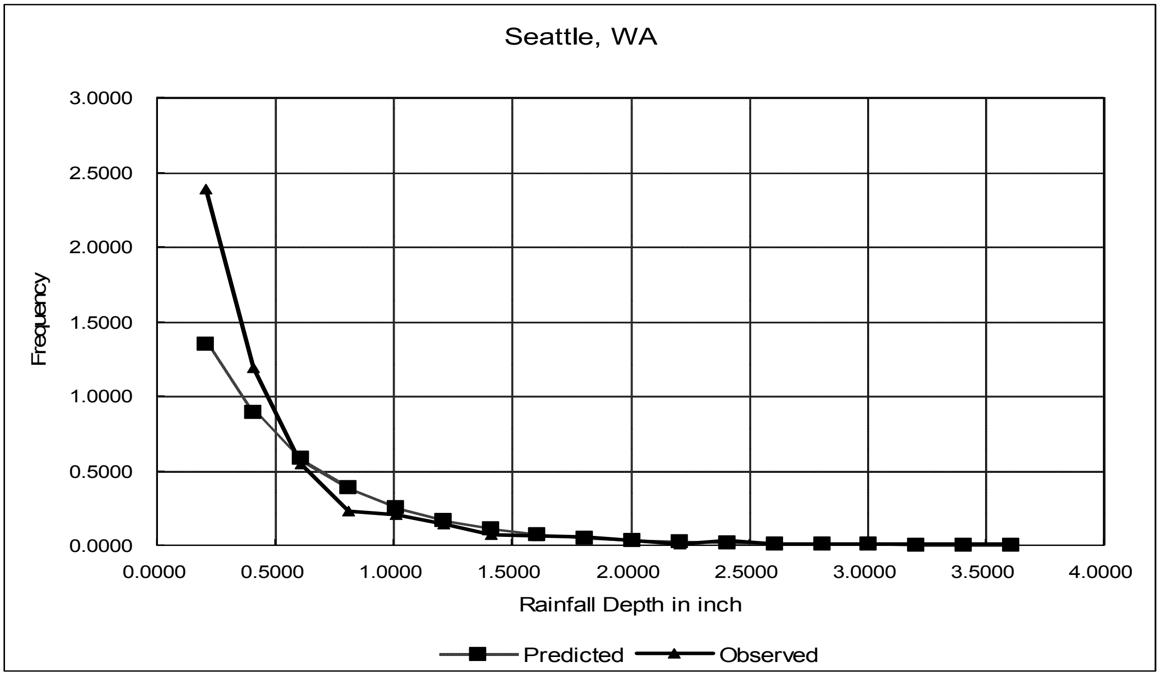

Figure 1 Seattle Rainfall Depth Distribution

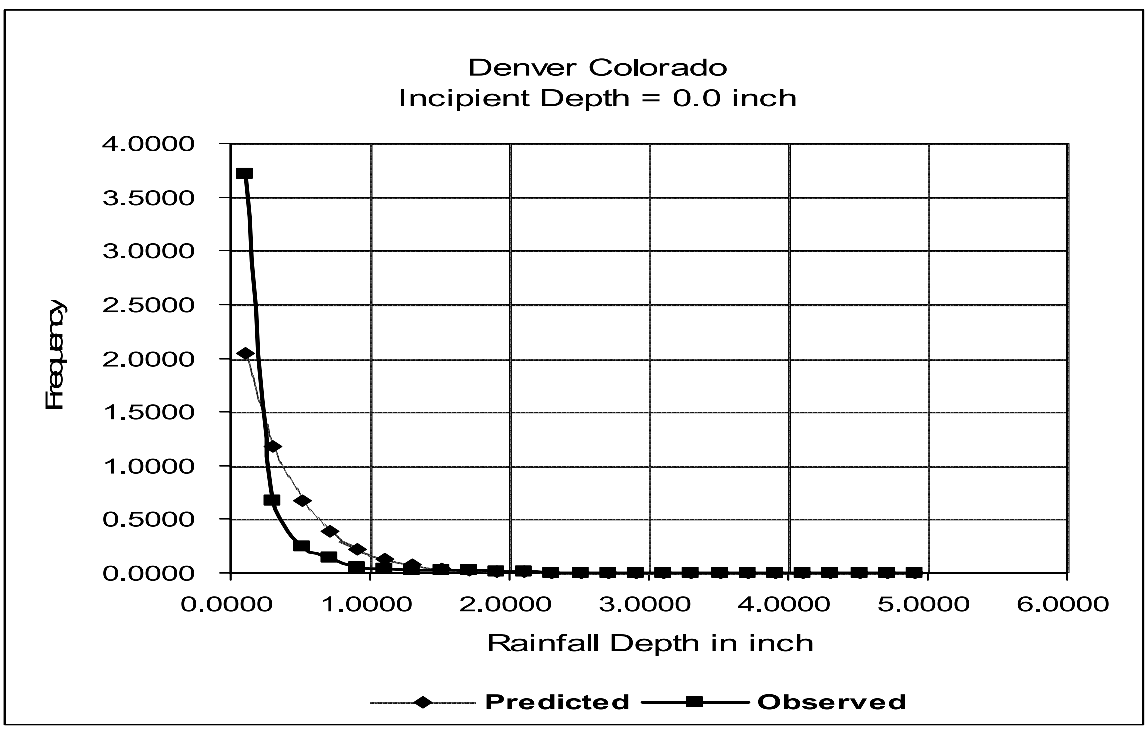

Figure 2 Denver Rainfall Depth Distribution

There are many recommendations on modeling the distributions of complete rainfall data series, such as exponential distribution (Bedient and Huber in 1992), one-parameter Poisson distribution (Wanielista and Yousef in 1993), and two-parameter model of Gamma distribution (Woolhiser and Pegram, 1979). In this study, the one-parameter exponential distribution is adopted to fit the frequency distribution of rainfall event depths. The exponential distribution is described as:

$$f(D) = \frac{1}{D_m} e^{\frac{-D}{D_m}} \quad (1)$$

in which f(D) = frequency of rainfall event-depth, D, and D_m = average rainfall event-depth.

Figures 1 and 2 present the observed and predicted distributions of rainfall event-depths for the two cities. For these sample data, the exponential distributions provide close agreements to the observed. For other cities and regions, the EPA report provides contour maps of average rainfall event-depths throughout the United States (EPA 1986). According to the Poisson process, the cumulative probability distribution of Eq 1 can be derived as:

$$P_D(0 \le d \le D) = 1 - e^{\frac{-D}{D_m}} \quad (2)$$

Eq 2 depicts the distribution of non-exceedance probability, P_D, that represents the chance to have an event-depth, d, not to exceed the design depth, D. The corresponding exceedance probability is

$$P_D(D \le d \le \infty) = e^{\frac{-D}{D_m}} \quad (3)$$

When selecting the storage capacity for a WQCB or the conveyance capacity for a trickle channel, Eq's 2 and 3 provide the basis to quantify the percentage of micro rainfall events to be captured by the facility or to overflow the facility.

RUNOFF CAPTURE CURVE

For convenience, the WQCV is often expressed in watershed mm. Since the purpose of this study is to determine the overflow risk for a selected WQCV, only runoff-producing events will be considered for analyses. As recommended, an incipient runoff depth of 2.5 mm is introduced to filter out extremely small rainfall events (Guo and Urbonas in 1996, Driscoll et al. in 1989). The WQCV of a basin can then be related to its design rainfall depth as:

$$V_o = C\,(D - D_i) \quad (4)$$

in which $V_o = WQCV$ in watershed mm, C= runoff coefficient, D = design rainfall depth in watershed mm, and D_i = incipient runoff depth in watershed mm. With D_i = 2.5 mm, approximately 20 to 30% of rainfall events are purged out of the rainfall series. For instance, the Denver rainfall record has a total of 1690 rainfall events. After filtering out the events less than or equal to 2.5 mm, there remains a total of 1263 rainfall events that have a potential of producing runoff from urban landscapes. Substituting Eq 4 into Eq 2 yields

$$\text{Cv} = P_D(0 \le V \le V_o) = P_D(0 \le d \le D) = 1 - ke^{\frac{-V_o}{CD_m}} \tag{5}$$

in which Cv = runoff volume capture rate, V_o = *WQCV* selected for design, $P_D(0 \le V \le V_o)$ = probability to have an event that produces a runoff depth less than V_o. The value of k is defined by the incipient runoff depth, and the average event rainfall depth as:

$$k = e^{\frac{-D_i}{D_m}} \tag{6}$$

The value of k varies in a narrow range between 0.80 and 0.90 among the seven cities used in this study. Eq 5 represents the synthetic runoff capture curve normalized by local average rainfall event-depth, runoff coefficient, and runoff incipient depth. In this study, Eq 5 is further tested as a one-parameter mathematical model to reproduce the simulated runoff capture curve under various urban landscaping conditions. The computer model, PONDRISK, was employed to divide a continuous rainfall record into individual events, and then to calculate the runoff capture curve for a specified runoff coefficient and incipient runoff depth (Guo 1992). The model, PONDRISK, has been used to produce the design runoff capture curves for the storm water design criteria used in the Denver metropolitan area, CO (Urban Storm Water Drainage Criteria Manual, 2001), and for Hydrology Standards used in the Sacramento metropolitan area, CA (Sacramento City/County Drainage Manual, 1996).

Figure 3 present comparisons between Eq 5 and the runoff capture curves produced by *PONDRISK* for Boston, MA, Seattle, WA, Sacramento, CA, and Phoenix, AZ using 20- to 30-year continuous hourly rainfall records. In general, Eq 5 provides good agreements to the long-term continuous records at these sample cities. Figure 4 presents a set of generalized runoff capture curves produced using Eq 5 with runoff coefficients of 0.2, 0.4, 0.6, 0.8, 0.9 and 1.0. It is noticed that the curvature of runoff capture curve increases when the runoff coefficient decreases. The runoff capture curve becomes almost a linear response between rainfall depth and runoff amount when C=1.0. This tendency reflects the fact that the higher the imperviousness in a catchment, the less the surface depression and detention. As a result, the response of a catchment to rainfall is quick and direct

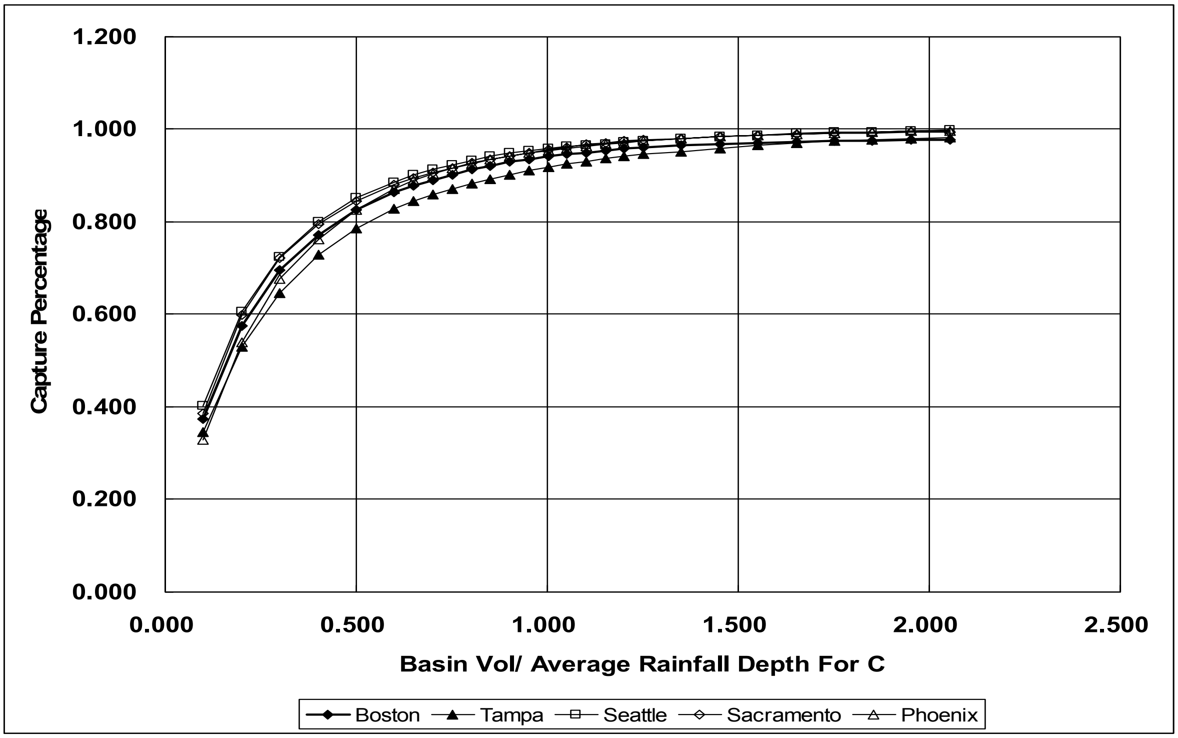

Figure 3 Comparison Among Normalized Runoff Capture Curves for C=0.5

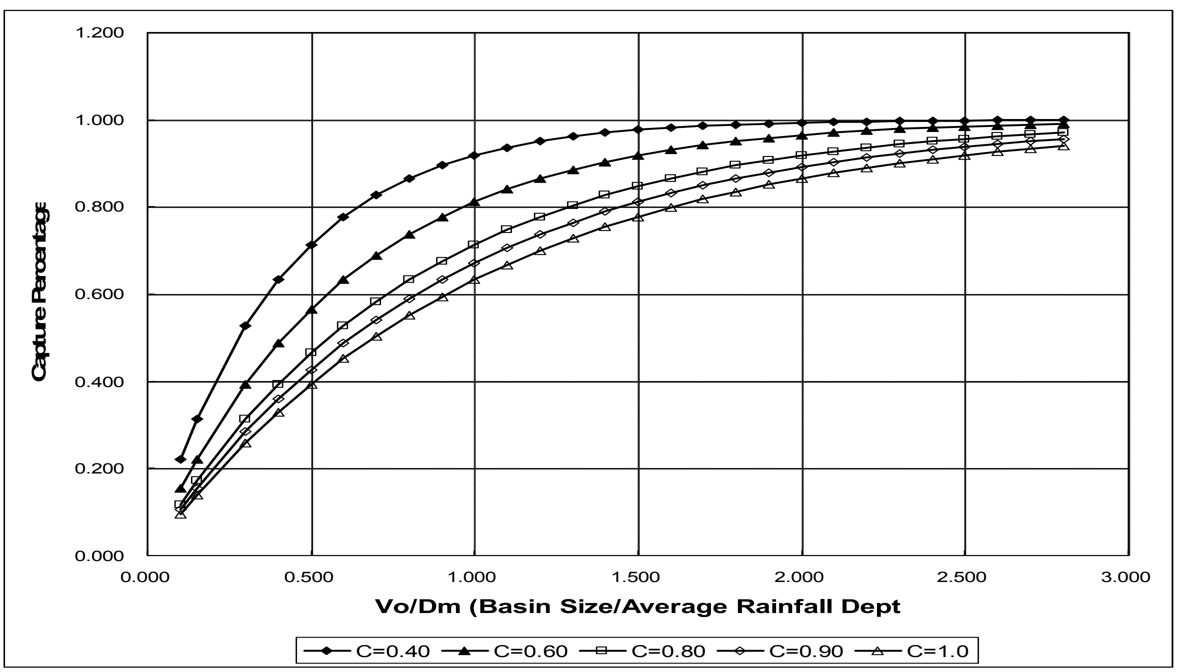

Figure 4 Normalized Runoff Capture Curves for Various Runoff Coefficients

The runoff capture curve provides important and necessary information when determining the WQCV for a WQCB design. In current practice, the runoff capture curve at a basin site is generated by runoff simulation techniques using a long term continuous rainfall record. Often, such a lengthy data process is not practical. Indeed, it is imperative that the design methodology be improved by the fundamental understanding of the distribution of runoff-producing rainfall series. Eq 5 provides a synthetic runoff capture curve by which the inherent overflow risk of a specified WQCV is calculated as:

$$R_e = P_D(D \le d \le \infty) = P_D(V_o \le V \le \infty) = ke^{\frac{-V_o}{CD_m}} \qquad (7)$$

in which R_e = inherent overflow risk. R_e varies between k when V_o=0 and zero when $V_o = \infty$. For a specified runoff coefficient, Eq 7 indicates that the inherent overflow risk decreases when the WQCV increases. The runoff coefficient also plays an important role in the decay factor in Eq 7. As expected, the overflow risk tends to be higher for a paved area than that for a pervious area.

RUNOFF DELIVERY CURVE

A trickle channel serves as a runoff collector through the tributary watershed and also acts as the low-flow channel cross section through the detention system. As a risk-based design, both a WQCB and its trickle channel should be designed on a consistent risk basis. However, the current practice is to assume that the capacity of a trickle channel is approximately 1 to 3% of the major design event (Urban Storm Water Drainage Criteria Manual, 1999). Such an empirical recommendation has not been examined by its hidden overflow risk, and does not provide consistent overflow risk for the design of a WQCB system.

As a common practice in the design of traditional drainage channels, an overflow risk should be pre-selected before determining the channel capacity. To apply such a concept to trickle channel designs, a runoff rate distribution shall be constructed to define non-exceedance probabilities versus channel capacities. In this study, the Rational method is employed to estimate the potential peak runoff after the storage capacity of the WQCB has been chosen from the runoff capture curve as:

$$Q_o = mCAI \qquad (8)$$

$$I = \frac{D}{T_c} \qquad (9)$$

in which Q_o = capacity of trickle channel in cms, m = unit conversion factor of 1/360, A= tributary watershed area in hectares, D = rainfall depth in mm, T_c = time of concentration in hours, and I = rainfall intensity in mm/hour. In this study, the conventional concept to assume time of concentration as design rainfall duration is adopted. In doing so, the highest rainfall intensity is applied to the entire tributary watershed. Substituting Eq 9 into Eq 8 yields:

$$D = \frac{T_c Q_o}{mCA} \tag{10}$$

Eq 10 is derived based on a consistent risk between the WQCB and its trickle channel. Substituting Eq 10 into Eq 2 yields:

$$C_Q = P_D(0 \le d \le D) = P_D(0 \le V \le V_o) = 1 - e^{\frac{-T_c Q_o}{mCAD_m}}$$
(11)

in which C_Q = percentage of runoff events that generate a runoff rate less than the capacity of the trickle channel. In this study, Eq 11 is termed the runoff delivery curve derived for trickle channel designs. A runoff delivery curve defines overflow risk versus channel capacity. The parameter in Eq 11 reflects the development of the tributary watershed in terms of C, and the drainage condition of the watershed in terms of T_c. Setting $Cv = C_Q$, Eq's 5 and 11 ensure that both the WQCB and its trickle channel can be designed for the same rainfall amount and subject to the same overflow risk.

DESIGN SCHEMATICS

To illustrate the design procedure, a WQCB located in Boston, MA is used as an example. The tributary watershed to the WQCB has a drainage area of 8098 square meters (2.0 acres or 0.81 hectare) and a runoff coefficient of 0.5. The time of concentration of the tributary watershed is calculated to be 20 minutes. In Boston, the average rainfall event depth is 17.78 mm, and the incipient runoff depth is 2.5 mm. Aided by Eq 5, the localized runoff capture curve for Boston is derived as:

$$Cv = P(0 \le d \le D) = 1 - 0.87e^{-0.1125V_o} \tag{12}$$

To target a non-exceedance probability of 78%, the value of V_o in Eq 12 is found to be 12.2 watershed mm or WQCV = 98.8 cubic meters. Similarly, substituting the design variables into Eq 11, the runoff delivery curve is derived as:

$$C_Q = P(0 \le d \le D) = 1 - e^{-16.66Q_o} \tag{13}$$

Applying the non-exceedance probability of 78% to Eq 13, the capacity of the trickle channel for this example is determined to be 0.041 cms. Both Eq's 12 and 13 are derived for the Boston area using the local rainfall average depth. Once the runoff capture percentage of 78% is selected, Eq's 12 and 13 consistently define the storage capacity for the basin and the conveyance capacity for the associated trickle channel.

The runoff capture curve varies between zero and unity. As shown in Figure 10, runoff capture curves are asymptotic to unity when V_o/D_m becomes large. In practice, care has to be taken to avoid infeasible solutions. It is advisable that a synthetic runoff capture curve be constructed for the target range of runoff volumes captured by the WQCB under design. For example, the WQCB shall be designed to capture 50.0 to 95.0 percent of runoff volume. Within this range, the engineer may take all design factors into consideration and quantify the associated overflow risk for each alternative. Figure 12 is the synthetic runoff capture curve generated for this example by Eq 12. The corresponding basin storage volume ranges from 4.92 to 25.40 watershed mm for the specified range of runoff capture percentage.

Within the target range, a tradeoff exists between basin storage volume and overflow risk. The traditional concept of "the larger, the better" is not economically justified for the design of WQCB. Therefore, a sensitivity study must be conducted to pinpoint the basin size for design. The objective is to select the basin size subject to its marginal return in runoff amount captured. Like many drainage designs (Guo 1998, and Guo 1999), Figure 12 exhibits a clear diminishing return of runoff captured when increasing WQCV. Mathematically, the optimization procedure requests that the tangent (local slope) at the proper basin size be equal to the average slope on the runoff capture curve for the target range. Figure 12 shows that for the range from 4.85 to 13.0 watershed mm, the percentage of runoff volume captured increases faster than the average return, and for the range from 13.0 to 25.3 mm, the increases in the runoff volume captured are less than the average return. The proper basin storage capacity for this case is then determined to be 13.0 watershed mm because its tangent on the runoff capture curve is equal to the average return for the selected range.

The above mathematical procedure can be formulated to directly solve for the proper basin size. Aided by Eq 5, the average slope on the runoff capture curve for the selected range is

$$S_a = k(e^{\frac{-V_1}{CD_m}} - e^{\frac{-V_2}{CD_m}})/(V_2 - V_1) \tag{14}$$

in which S_a = average return or slope, V_1 = low limit, and V_2 = upper limit. The tangent on the runoff capture curve is equal to the first derivative of Eq 5 as:

$$S_o = \frac{\mathrm{k}}{CD_m} e^{\frac{-V_o}{CD_m}} \tag{15}$$

in which S_o = tangent on runoff capture curve. Setting Eq 14 equal to Eq 15, the proper basin size is determined as:

$$V_0 = -CD_m \ln\left\{\frac{CD_m}{(V_2 - V_1)}\left[e^{\frac{-V_1}{CD_m}} - e^{\frac{-V_2}{CD_m}}\right]\right\} \tag{16}$$

To apply Eq 16 to the case study, V_1 = 4.92 watershed mm, and V_2 = 25.4 watershed mm. By Eq 14, the average return, S_a , is 0.022. The proper basin size is found to be 13.2 watershed mm by Eq 16, and its runoff capture rate is 80.5 %, according to the local runoff capture curve, i.e. Figure 5. This procedure pinpoints the optimal basin storage capacity according to the target range. For example, the optimal basin size for this case will change to 12.44 watershed mm when the target range is set to be between 40% to 95%, or to 14.83 watershed mm when the target range is set to be between 60% to 95%.

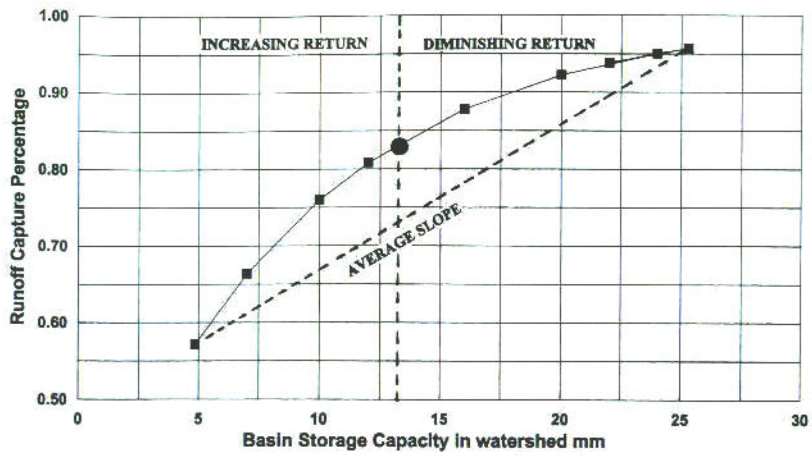

Figure 5 Illustration of Maximized Water Quality Control Volume

CONCLUSIONS

(a) Storm water quality control facilities are designed to improve storm water quality by reducing the pollutants carried by stormwater. The proper sizing of a WQCB for the runoff volume it will capture and treat is very important. The capacity of a WQCB shall be sized to capture the overwhelming fraction of rainfall events referred to in this paper as micro rainfall events, namely, not the extreme events used in traditional urban drainage practice for flood control purposes. In this study, the complete rainfall data series derived from 20- to 30-year continuous records were analyzed for seven metropolitan areas in the United States. Results indicate that the distribution of rainfall event-depths can be described as an exponential distribution. Further, the integration of the exponential distribution provides the non-exceedance probability distribution for rain event-depth.

(b) This study presents a methodology to directly synthesize the runoff capture curve for any basin site using the local average rainfall event-depth. This technique has been examined by long-term rainfall data recorded in seven major metropolitan areas in the United States. Close agreements have been observed between the synthetic runoff capture curve and derived runoff capture curve from deterministic modeling using continuous rainfall records. Knowing the runoff capture curve, this study recommends an optimization procedure to identify the proper basin size using the concept of diminishing return.

(c) The trickle channel associated with a WQCB shall be designed with a consistent overflow risk. This study presents a methodology to synthesize the runoff delivery curve by which the relationship between channel capacity and non-exceedance probability can be defined. The method provides a basis to warrant a consistent overflow risk used in the design of a BMP system which involves trickle channels and WQCB.

(d) To apply this method to determine the WQCV for the design of a WQCB in the continent of the United States, the required prior knowledge of average rainfall event depths is presented in Figure 6. Details can be found elsewhere. (EPA 1986)

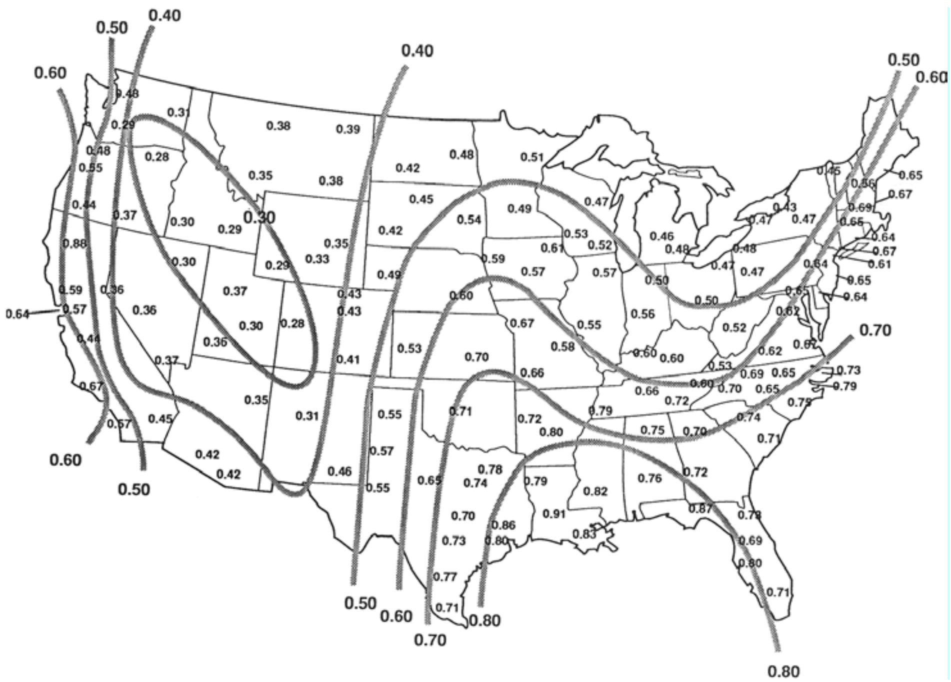

Figure 6 Event Average Rainfall Depth for USA

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Definition of Equation Symbols

A = tributary watershed area
C = runoff coefficient
Cv = runoff capture rate by a detention basin
C_Q = runoff delivery rate by a trickle channel
d = random variable for rainfall depth
D = design rainfall depth for detention basin
D_1 = limit for rainfall depth
D_m = average rainfall depth
f(t) = Probability Density Function
m = unit conversion factor
k = variable defined by the ratio of runoff incipient depth to local average event depth.
$P_D(0 \le d \le D) = P_D(0 \le V \le V_o)$ = Cumulative Probability from 0 to D or from 0 to V_o
Q_o = design capacity for trickle channel
R_e = overflow risk
SD = standard deviation
S_a = average slope on runoff capture curve
S_o = tangent on runoff capture curve
T_c = time of concentration
V_o = basin storage volume
WQCV = water quality control volume in mm per watershed
WQCB = water quality basin