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# OVERFLOW RISK ANALYSIS FOR STORM WATER QUALITY CONTROL BASINS

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***Abstract***

The operational cycle of a storm water quality control basin can be divided into the waiting period between events and the filling and draining period during an event. In this study, an inherent overflow risk is defined as the probability of having a large event exceed the basin storage capacity. Such a probability is prescribed by the basin storage capacity and the local distribution of rainfall event depths. An operational overflow risk is defined as the probability of having the basin overwhelmed by a subsequent storm event during the draining process. An operational risk is found to be dependent on watershed runoff coefficient, basin drain time, local average rainfall event depth, and average rainfall interevent time. In practice, the selection of a basin drain time is a tradeoff between the removal of suspended solids in storm water and the overflow risk. The concept of "the longer, the better "applies to the sedimentation process, but concern for the overflow risk requires that the basin drain as fast as possible. This paper presents a design method by which the overflow risk associated with a basin storage volume can be evaluated for various drain times. The mathematical models developed to describe the distribution of rainfall interevent time and the runoff capture curve provide good agreement with the long-term continuous rainfall data recorded in seven metropolitan areas in the United States. The risk-based approach developed in this study provides a quantifiable basis for making the decision on the operation of a storm water quality control basin.

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

## INTRODUCTION

Urban drainage facilities are designed using risk-based approaches. According to the pre-selected risk, the proper capacity is then determined for the drainage structure. For instance, to design a minor or major flood control facility, the probabilistic distributions for extreme events are employed to prescribe the relationship between design capacity and overflow risk (US Water Resources Council 1981). Similarly, to design a storm water quality control basin (WQCB), such a relationship is prescribed by the runoff capture curve (Guo and Urbonas 1996, Guo and Hughes 2001, Mays 2001). The challenge in the design of a WQCB is the tradeoff between design variables. For instance, a short drain time is preferred in order to reduce the overflow risk while the basin is emptying out. However, from the sediment settlement and pollutant removal perspective, it is necessary to have a long drain time. The question arises as to how to quantify the overflow risk for the selected water quality control volume (WQCV) and its drain time. To improve the current design methodology for determining a WQCV, it is necessary to understand the distribution of a complete rainfall data series and the relationships among WQCV, drain time, and overflow risk.

In this study, the continuous rainfall records from seven major cities located in the United States were analyzed by individual storm events. An investigation was conducted on the distributions of rainfall event depths and interevent time. An exponential distribution was adopted as the mathematical model to represent the distributions of rainfall event depth and interevent time derived from a complete rainfall series. A design methodology was then developed to evaluate the relationship between WQCV and overflow risk using the local rainfall distribution for various combinations of drain time and WQCV.

## DISTRIBUTIONS OF MICRO RAINFALL EVENTS

To assess the overflow risk of a WQCB, it is necessary to know the local frequency distributions of rainfall event depth and interevent time. Rainfall frequency analysis is often referred to as an analysis of rainfall intensity versus rainfall duration for various recurrence intervals. Examples are HYDRO 35 and TP40 (Frederick et al. 1977; Hershfield 1961). However, in this study, the rainfall frequency analysis is referred to as a histogram analysis. For instance, to study the frequency distribution of rainfall interevent times, the 20- to 30-year hourly continuous rainfall records were examined for the following seven metropolitan cities: 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 using a storm separation time of six hours and the procedures recommended in EPA studies (EPA Report 1986). Details of these rainfall data analyses can be found elsewhere (Guo and Urbonas 1996). These histogram analyses indicate that a large number of rainfall events have a short interevent time and the frequency of occurrence decreases as the interevent time increases. Similarly, a large number of rainfall events have a small rainfall event depth and the frequency of occurrence decreases as the rainfall event depth increases. Many models have been developed to describe the distribution of complete rainfall series, such as the exponential distribution (Bedient and Huber 1992), the one-parameter Poisson distribution (Wanielista and Yousef 1993), and the two-parameter model of gamma distribution (Woolhiser and Pegram 1979). In this study, the one-parameter exponential function was tested and then adopted to describe both the frequency distributions of rainfall interevent time and event depth. The exponential function for rainfall interevent time is formulated with its mean value as:

$$f(T) = \frac{1}{T_m} e^{\frac{-T}{T_m}} \qquad (1)$$

in which *f(T)* = frequency for rainfall interevent time, T, and $T_m$ = average rainfall interevent time. For example, applied a 6-hour rainfall event separation time to the long-term rainfall records, Table 1 summarizes the rainfall statistics for seven metropolitan cities in the United States. Figures 1 and 2 present the observed and predicted rainfall distributions for Denver and Boston. In general, the exponential distribution provides close agreement with the rainfall data used in this study.

| | Interevent Time | | | Rainfall Depth | | |
|---|---|---|---|---|---|---|
| City | Mean $T_m$ hours | Standard Deviation SD Hours | Skewness Coeff $C_s$ | Mean $D_m$ Mm | Standard Deviation SD mm | Sknewness Coeff $C_s$ |
| Seattle, WA | 53.52 | 87.36 | 4.42 | 12.45 | 12.42 | 2.75 |
| Sacramento, CA | 166.71 | 387.51 | 4.92 | 15.49 | 15.75 | 2.96 |
| Phoenix, AZ | 261.32 | 422.25 | 3.47 | 10.67 | 9.12 | 2.59 |
| Denver, CO | 106.42 | 130.89 | 2.62 | 10.41 | 11.94 | 3.59 |
| Cincinnati, OH | 65.23 | 71.92 | 2.47 | 14.73 | 13.97 | 3.03 |
| Tampa, Florida | 71.42 | 99.75 | 3.76 | 16.76 | 19.81 | 4.40 |
| Boston, MA | 70.65 | 67.48 | 1.90 | 17.78 | 20.07 | 4.98 |

**Table 1. Statistics of Inter-event Time and Rainfall Depth Using 6-hr Separation Time**

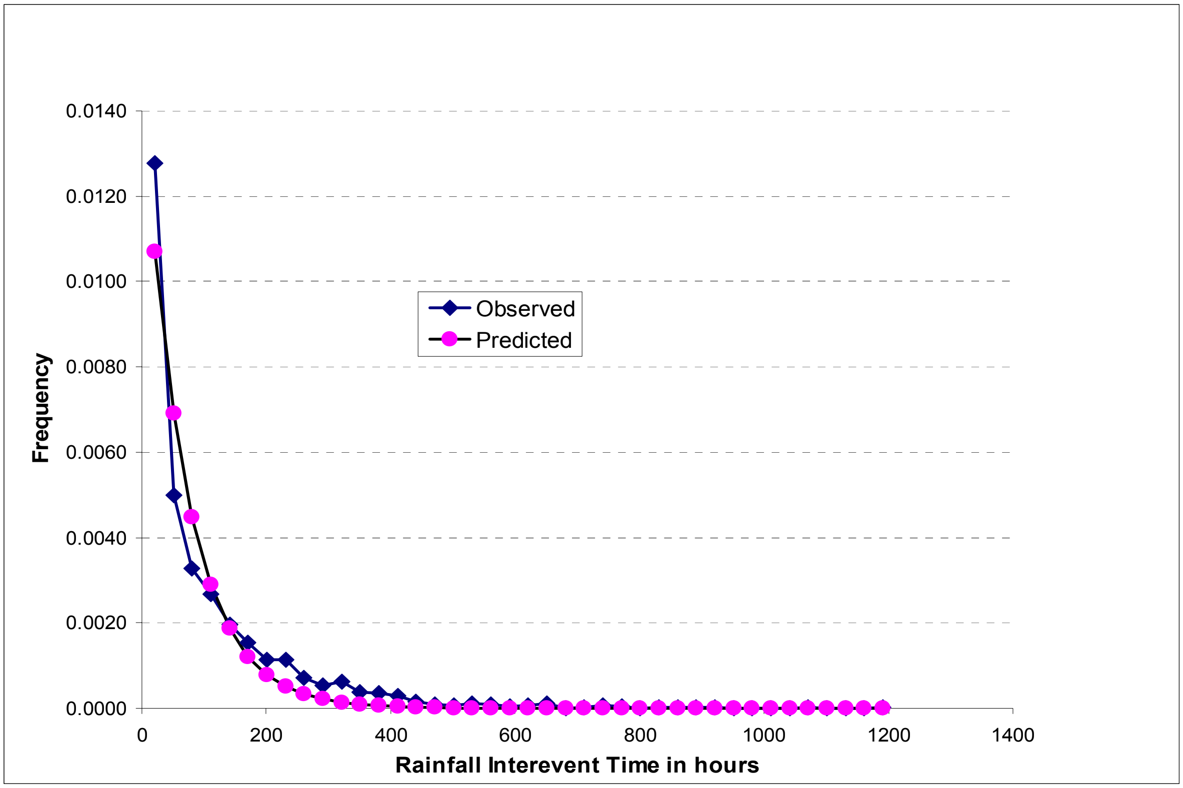


**Figure 1 Distribution of Inter-Event Time For Denver, Colorado**

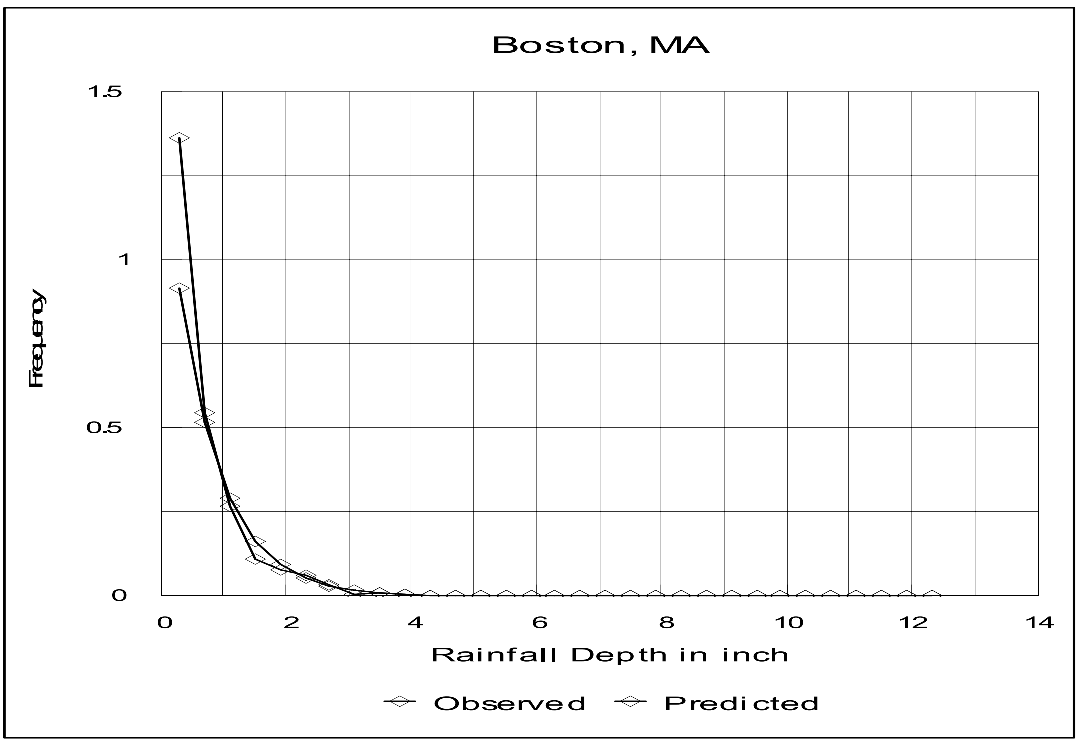


**Figure 2 Rainfall Event Depth Distribution for Boston, MA**

The frequency distribution serves as the density function for a probability distribution. The cumulative probability for an event to occur during a period of time is an integration of Eq 1 as:

$$P_T(T_1 \le t \le T) = \int_{T_1}^{T} \frac{1}{T_m} e^{\frac{-t}{T_m}} dt = e^{\frac{-T_1}{T_m}} - e^{\frac{-T}{T_m}} \qquad (2)$$

in which $P(T_1 \le t \le T)$ = probability to have an event in the time period between $T_1$ and T, and t = time variable. When $T_1 = 0$, the probability in Eq 2 is the chance to have an event during the elapsed time, T, as

$$P_T(0 \le t \le T) = 1 - e^{\frac{-T}{T_m}} \qquad (3)$$

Also, Eq 3 is the non-exceedance probability representing the chance to have the next event within a waiting time, T. Correspondingly, the exceedance probability for the selected interevent time is:

$$P_T(T \le t \le \infty) = e^{\frac{-T}{T_m}} \qquad (4)$$

Similarly, the exponential distribution can also be applied to the distribution of rainfall event depth as:

$$f(D) = \frac{1}{D_m} e^{\frac{-D}{D_m}} \qquad (5)$$

in which D = rainfall event depth, and $D_m$ = average rainfall event depth. In this study, Eq 5 was tested by long term rainfall records. To apply Eq's 1 and 5 to the cities and regions in the United States, the EPA study provides contour maps of the average rainfall interevent time (EPA report 1986).

In practice, the overflow risk of a WQCB is more directly related to runoff depths than rainfall depths. Therefore, the rainfall depth of an event is converted to its runoff depth as:

$$V = C(D - D_i) \quad \text{if } D > D_i \qquad (6)$$

in which V = runoff depth in mm which produces a volume in mm per watershed, C= runoff coefficient used in the Rational method, D = rainfall event depth in mm, and $D_i$ = incipient runoff depth in mm. An incipient runoff depth is the minimum amount of rainfall depth for producing runoff. Recognizing that the incipient runoff depth varies with respect to land use, the EPA study recommends an average incipient runoff depth of 2.5 mm for urban areas (EPA report 1986). When converting rainfall to runoff depth, events that produced a rainfall depth less than the incipient runoff depth should be purged out from the data base (Guo and Urbonas, 1996). Aided by Eq 5, Eq 6 is integrated as:

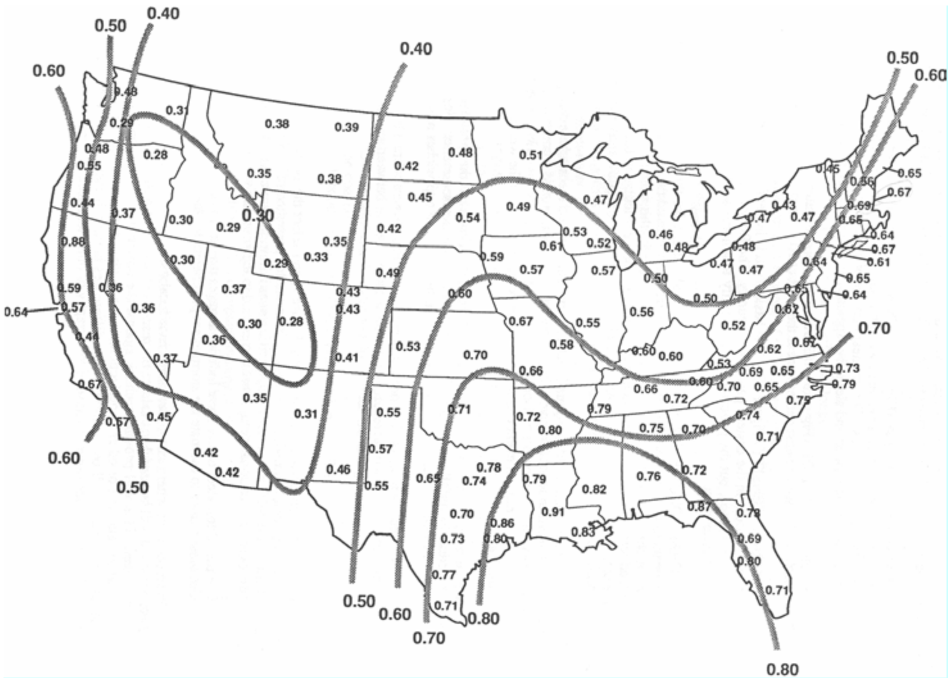


**Figure 3 Event Depth for the United States**

$$Cv = P_D(0 \le V \le V_o) = P_D(0 \le d \le D) = 1 - ke^{\frac{-V_o}{CD_m}} \quad (7)$$

$$k = e^{\frac{-D_i}{D_m}} \quad (8)$$

in which $C_v$ = non-exceedance probability for having a rainfall event which produces a runoff depth, V, less than the referenced depth, $V_o$. For convenience, all rainfall volumes, runoff volumes, and basin storage capacities used in this study are expressed in millimeters per watershed. For a WQCB designed with a WQCV equal to $V_o$, Eq 7 represents its long-term runoff capture percentage. For example, with C = 0.5, Figure 6 presents the comparison between Eq 7 and the runoff capture curves derived from continuous rainfall/runoff simulations using long-term continuous rainfall data recorded in five cities. Variations in Figure 6 might have been caused by the assumption of $D_i$ = 2.5 mm for all meteorological regions. For instance, the incipient runoff depth for Phoenix, AZ should be higher than that for Tampa, Florida. More studies are needed to understand the variations of incipient runoff depth and its sensitivity to the runoff capture. According to the EPA study (EPA Report 1986), the value of k in Eq. 8 generally varies between 0.70 and 1.0 for metropolitan areas.

In practice, a flood control detention system is designed for a pre-selected recurrence interval using a flood-frequency curve derived from extreme series while a WQCB is designed to capture a target percentage of runoff volume defined in terms of runoff depth by Eq 7. Eq 7 is termed the runoff capture curve (Guo and Hughes 2001, Mays 2001, and Guo 2001). Both the flood-frequency curve and the runoff capture curve define the *inherent overflow risk* for the selected design event. The non-exceedance probability of a flood control detention basin is referenced to the recurrence interval of the design flood, and the non-exceedance probability of a WQCB is defined by the percentage of runoff amount captured. Eq 7 indicates that the runoff capture curve depends on the runoff coefficient of the watershed, the on-site rainfall event depth and incipient depth, and the selected WQCV. As an example for sensitivity study, the normalized runoff capture curves in Figure 7 were produced with various runoff coefficients under the assumption of k = 1.0. As expected, the runoff capture curves vary with respect to runoff coefficients. In general, the curvature of runoff capture curve decreases when the runoff coefficient increases. It implies that the runoff capture percentage for a WQCV decreases when the imperviousness of the catchment increases.

## OVERFLOW RISK

The overflow risk of a WQCB is defined as the probability of having a rainfall event that produces a runoff volume more than the available storage capacity in the WQCB. The operational cycle of a WQCB is divided into two periods*: (1) the waiting period between events, and (2) the draining period during an event.* After a basin is completely drained, its design WQCV, or $V_o$ in Eq 7, is 100% available for the next event. During a waiting period, the overflow risk for the WQCB can be formulated based on the scenario that the WQCB is to be overwhelmed by a single large incoming event. During an event, the basin is operated under the dynamic processes of filling and draining and the WQCV is partially or fully occupied by the event runoff. Therefore, likelihood exists that only partial WQCV is available for the next event. This fact adds an additional overflow risk to the WQCB during its draining process. An operational overflow risk is subject to the available storage volume in the basin versus the magnitude of the subsequent event. Mathematically, the assessment of overflow risk for both waiting and draining periods can be separately derived under the different conditions.

## INHERENT OVERFLOW RISK

During a waiting period, the overflow risk for an empty basin is a function of the waiting time, T, and can be defined by the two following conditions: (1*) a rainfall event will come within the waiting time, T, and* (2) *the next event will produce a runoff volume exceeding the WQCV .* Such a joint probability is calculated as:

$$R_e(0 \le t \le T) = P_T(0 \le t \le T) * P_D(V_o \le V \le \infty) \quad (9)$$

in which $R_e$ =overflow risk when the basin is empty, $T$ = waiting time, $P_T$ = probability for interevent time, $P_D$ = probability prescribed by the runoff capture curve, V = runoff volume in mm per watershed, and $V_o$ = WQCV in mm per watershed. Aided by Eq's 3 and 7, Eq 9 becomes

$$R_e(0 \le t \le T) = (1 - e^{\frac{-T}{T_m}})ke^{\frac{-V_o}{CD_m}} \quad (10)$$

When the waiting time becomes long enough, Eq 10 is reduced to

$$R_e = ke^{\frac{-V_o}{CD_m}} \quad (11)$$

Eq 11 represents the *inherent overflow risk* for a WQCV equal to $V_o$. For a given catchment, the inherent overflow risk of a WQCB depends on the ratio of its WQCV to the local average rainfall event depth.

## OPERATIONAL OVERFLOW RISK

Once the basin is full by an event, the overflow risk is subject to the potential available storage volume and the magnitude of the next incoming event. During the draining process, the available storage volume in the basin increases as the elapsed time, *T*, increases and can be calculated as:

$$V(T) = q\,T \quad \text{for} \quad T<T_d \tag{12}$$

in which $V(T)$ = available storage volume in mm per watershed at elapsed time $T$, $q$ = average release rate from the WQCB in mm/h, and $T_d$ = drain time in hours. Recognizing that the release from a basin is a hydrograph, the average release can be related to the peak release (Guo 1999). When $T = T_d$, the average release is defined as:

$$q = \frac{V_o}{T_d} \tag{13}$$

The overflow risk during the draining period from $T$ to $T_d$ depends on the two probabilities: (1) *the next event will come between T and $T_d$, and (2) the next event will produce a volume exceeding the available storage volume*. Such a joint probability can be formulated as:

$$R_D(T \le t \le T_d) = P_T(T \le t \le T_d) * P_D(V(T) \le V \le \infty) \tag{14}$$

in which $R_D$ = overflow risk during the draining process. Aided by Eq's 2 and 7, Eq 14 becomes

$$R_D(T \le t \le T_d) = (e^{\frac{-T}{T_m}} - e^{\frac{-T_d}{T_m}})ke^{\frac{-qT}{CD_m}} \tag{15}$$

Eq 15 describes the *operational overflow risk* that is caused by a subsequent rainfall event during the draining process. As expected, the operational overflow risk decreases as the elapsed time increases and vanishes when $T = T_d$.

## TOTAL OVERFLOW RISK FOR A CYCLE OF OPERATION

Consider a cycle of operation beginning with an empty basin. During the waiting period, the basin is subject to the inherent overflow risk, i.e. overwhelmed by a single large event. During the draining period, the basin is subject to the additional operational overflow risk by a subsequent event before the basin is empty. Therefore, the total overflow risk is equal to the sum of Eq's 11 and 15 as:

$$R(T) = R_e + R_D \qquad \textit{for } 0 \le T \le T_d \tag{16}$$

in which $R(T)$ = total overflow risk at elapsed time $T \le T_d$. Substituting Eq's 11 and 15 into Eq 16 yields:

$$R(T \le t \le T_d) = ke^{\frac{-V_o}{CD_m}} + (e^{\frac{-T}{T_m}} - e^{\frac{-T_d}{T_m}})ke^{\frac{-qT}{CD_m}} \qquad \text{for } 0 \le T \le T_d \tag{17}$$

Eq 14 indicates that *R(T)* has its highest value at *T* = 0 when the basin is full, and the lowest value at $T = T_d$ when the basin is empty. Substituting *T* = 0 into Eq 17 yields:

$$R(0) = ke^{\frac{-V_o}{CD_m}} + k(1 - e^{\frac{-T_d}{T_m}}) \quad \text{at T=0} \quad (18)$$

Eq 18 is the overflow risk at the beginning of the draining process when the basin is nearly full. Substituting $T = T_d$ into Eq 18 yields

$$R(T_d) = ke^{\frac{-V_o}{CD_m}} \quad \text{for } T \geq T_d \quad (19)$$

Eq. 17 begins with *R(0)* prescribed by Eq 18 and then converges to $R(T_d)$ by Eq 19 as the elapsed time, *T*, increases. After $T_d$ the entire WQCV in the basin becomes available, the overflow risk is therefore reduced to its inherent risk, i.e. $R(T_d)$ and another cycle of operation will begin.

## DESIGN SCHEMATICS

To illustrate the design procedure, a WQCB located in Boston, MA is used as an example. The tributary watershed has a drainage area of 8098 square meters (2.0 acres) with a runoff coefficient of 0.5. At Boston, the average rainfall event depth is 17.78 mm and the average interevent time is 70.65 hours. Considering that a runoff incipient depth of 2.5 mm, the value of k is 0.86 for the Boston area.

The design WQCV was determined to be 13.2 mm per watershed. Based on the characteristics of sediments found in the local storm water runoff, the drain time required to remove most of sediments was determined to be 24.0 hours. With $V_o$ = 13.2 mm and a drain time $T_d$ = 24 hours, the average release rate from the basin was determined by Eq 13 as:

$q = V_o / T_d$ = 13.2/24.0 = 0.55 *watershed mm* per hour (20)

According to Eq 11, the inherent overflow risk for an empty basin was

$$R_e = 0.86e^{\frac{-13.20}{0.50*17.78}} = 0.195 \quad for \quad T \geq T_d \quad (21)$$

Substituting Eq 21 with $D_m$ = 17.78 mm and $T_m$ = 70.6 hours into Eq 17 yields

$$R(T) = 0.195 + 0.86(e^{\frac{-T}{70.65}} - e^{\frac{-24.0}{70.65}})e^{\frac{-0.55T}{0.50*17.78}} \quad for \quad 0 \leq T \leq T_d \quad (22)$$

According to Eq 18, the upper limit of Eq 22 is defined by T=0 as:

$$R(0) = 0.195 + 0.86(1 - e^{\frac{-24.0}{70.65}}) = 0.447 \quad (23)$$

Eq 22 indicates that the overflow risk during the drain time is reduced from 0.447 to 0.195 as the elapsed time, *T*, increases.

From the sedimentation aspect, a longer residence time is preferred because it captures more sediment. But on the other hand, it also introduces a higher overflow risk to the basin operation. In practice, a range of drain times may be selected by the sedimentation requirements. Each drain time can be calculated by the associated overflow risk. This process assists the engineer in making a final selection for the drain time based on the trade off between the overflow risk and the amount of sediment captured. For example, using WQCV=13.2 mm per watershed and C=0.5, Table 2 presents the variations of the overflow probabilities for drain times of 12-, 24-, 48-, 72-, and 96-hour. For a selected drain time, the overflow risk begins with its highest level when the basin is full, and then gradually reduces through the emptying process. After the basin becomes empty, the overflow risk converges to the inherent risk level determined by the basin storage capacity. Among various drain times, as expected, the longer the drain time, the higher the overflow risk. In design, it is important to know the associated overflow risk for the selected drain time. A longer residence time enhances the sedimentation process for the event, but induces a higher overflow risk to the next event. Therefore, the selection of a proper drain time should take sedimentation enhancement into consideration, including treated, partially treated, and untreated runoff volumes. Nevertheless, the methodology developed in this study provides a simplified mathematical model to synthesize the localized runoff capture curve with the overflow risk for the selected drain time.

| Elapsed | | Overflow | Risk for | Drain | Time |
|---|---|---|---|---|---|
| hour | 12-hr | 24-hr | 48-hr | 72-hr | 96-hr |
| 0.00 | 0.332 | 0.447 | 0.626 | 0.752 | 0.841 |
| 6.00 | 0.226 | 0.320 | 0.493 | 0.624 | 0.718 |
| 12.00 | 0.195 | 0.250 | 0.397 | 0.523 | 0.617 |
| 24.00 | 0.195 | 0.195 | 0.280 | 0.380 | 0.466 |
| 36.00 | 0.195 | 0.195 | 0.222 | 0.294 | 0.365 |
| 48.00 | 0.195 | 0.195 | 0.195 | 0.242 | 0.297 |
| 60.00 | 0.195 | 0.195 | 0.195 | 0.212 | 0.253 |
| 72.00 | 0.195 | 0.195 | 0.195 | 0.195 | 0.224 |
| 84.00 | 0.195 | 0.195 | 0.195 | 0.195 | 0.206 |
| 96.00 | 0.195 | 0.195 | 0.195 | 0.195 | 0.195 |

**Table 2 Overflow Risk for Various Drain Times**

## CONCLUSIONS

(a) In this study, the exponential distribution was tested to describe the distributions of rainfall interevent time and event depth. The runoff capture curve derived from the exponential distribution provides the non-exceedance probability distribution by which the overflow risk for the WQCB can be calculated based on the basin storage volume, watershed runoff coefficient, and local average rainfall depth and interevent time.

(b) The overflow risk of a WQCB was formulated by inherent risk and operational risk. An inherent overflow risk depends on the ratio of the basin storage capacity to the local average rainfall event depth, and an operational overflow risk depends on the ratio of the basin drain time to the local average rainfall interevent time.

(c) To design a WQCB, the following procedure is recommended: (1) to develop the local runoff capture curve at the basin site, (2) to select the design variables, including WQCV and drain time according to the local sediment or pollutant characteristics in storm water runoff, and

(d) to evaluate the overflow risk for the WQCB operation. The methodology developed in this study provides a consistent and quantifiable basis to calculate the overflow risk among different combinations of design variables. The required average rainfall interevent time and event depth can be found in the 1986 EPA report for the United States.

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