ANALYSIS OF LONG TERM RAINFALL TO DEVELOP STORMWATER QUALITY FLOW RATE DESIGN CRITERIA

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ABSTRACT

The intent of stormwater quality best management practices (BMPs) is to preserve and/or improve the existing quality of our water resources by achieving a water quality outcome. The water quality outcome is often expressed as a desired level of annual total suspended solids (TSS) removal and by a minimum volume of runoff that must be stored called the water quality volume (WQV), (USEPA, 2004). Traditional structural BMPs such as wet ponds and wetlands, are designed based on the WQV which is defined by a depth of rainfall that represents treating the 80th to 90th percentile volume of the annual runoff. However, the design basis for the WQV is often applied generically to all types of stormwater management BMPs without considering unit process limitations to manufactured BMPs.

Manufactured BMPs are intended for water quality improvement and are designed for space constrained sites. The design criteria for manufactured BMPs are flow based and performance is dependent on a prescribed particle size gradation. Therefore, applying the WQV design methodology to manufactured BMPs would be contrary to a flow based design principle and problematic when land for large detention facilities is not available. Flow based sizing, however does not mean treatment of design storms (i.e., 2 to 100 year storms which are typically applied for quantity control design) but rather treating the flows that contribute to the majority of the average annual runoff volume,

The purpose of this paper is to propose a flow based sizing methodology for manufactured BMPs called the water quality flow (WQF). An overview of the "first principles" used to determine a WQV is discussed and a similar approach to determine a WQF is proposed. Like the WQV, the WQF is selected based on treating the 80th to 90th cumulative percentile volume of the annual runoff.

INTRODUCTION

Traditional structural BMPs, such as ponds and wetlands, are designed to store and treat a minimum volume of runoff. This minimum volume is often referred to as the Water Quality Volume (WQV). The WQV is defined as the depth of rainfall over an impervious area that yields the minimum volume of detention required in a treatment facility. The objective of the depth of rainfall, or the WQV, is to treat 80% to 90% of the average annual volume of runoff.

In order to apply the WQV to flow based technologies, a flow rate is determined based on the return event that is most representative of the WQV. As an alternative, a method for determining a design flow rate based on the WQV is proposed. The flow rate is determined by analyzing local continuous historical rainfall records to create hydrographs for all the events. With the hydrographs, the design flow rate that yields an annual volume of runoff that best represents or equates to the WQV is selected. The results of the analysis will yield a flow rate that meets the objective of the WQV, i.e. 80% to 90% of the annual volume of runoff is treated. This flow rate is referred to as the Water Quality Flow Rate (WQF).

BACKGROUND

A method for determining the WQV was established in the "Selection and Design of Passive Treatment Controls", published by the ASCE and WEF, and further issued as guidance by the United States Environmental Protection Agency (USEPA, 2004). The methodology was developed for the practice of selecting a WQV based on the analysis of local historical rainfall data. Through the analysis, WEF, ASCE and the USEPA recognized that small rainfall storms (20 mm to 36 mm) naturally dominate the volume of runoff accumulated on an annual basis and contribute to the majority of the annual pollutant loading. ASCE and WEF published the results of a study of six detention basins in the United States (Roesner et al., 1991) that were tested for runoff capture by comparing efficiencies versus detention storage. Using continuous simulation, the six sites were modeled for flow capture and a relationship of annual runoff capture versus annual volume was developed (Figure 1).

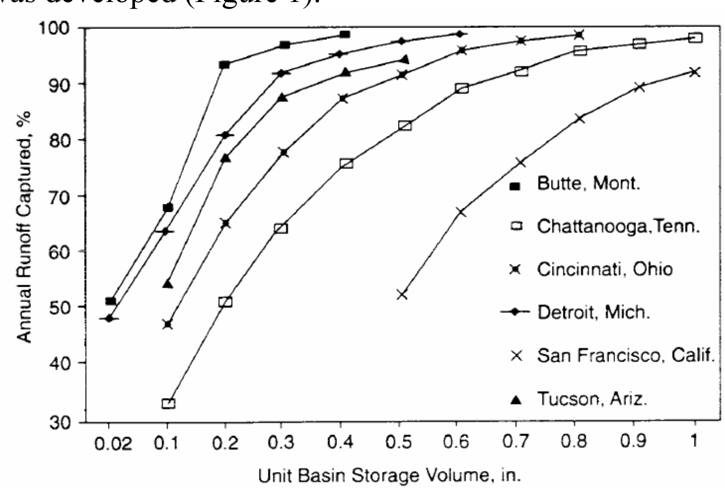

Figure 1. Runoff capture rates versus unit storage volume at six study sites (Roesner et al., 1991)

By finding a relationship between the annual runoff capture and volume of rainfall depth, the WQV can be determined. The results of the analysis concluded that the most cost-effective basin size is located at the "knee of the curve" (ASCE and WEF, 1998). The ASCE and WEF (1998) defines the "knee" as the maximum optimal volume to be treated. The economic return rapidly diminishes after this point since the infrequent large storms do not significantly contribute to the to the total annual rainfall volume.

Further analysis was conducted by the ASCE and WEF (1998) to assess the sensitivity of TSS removal based on the volume of a basin. The sensitivity analysis concluded that doubling the size of a treatment facility yield little gain in the amount of volume treated and TSS removal efficiency (ASCE and WEF, 1998). In addition, decreasing the basin volume by 30% showed very little decrease in the volume of runoff captured and TSS removal efficiency (ASCE and WEF, 1998). Examples of WQV design criteria currently used in North America are listed in Table 1.

Table 1. Regulatory Water Quality Volumes by Province and State

Regulatory Agency	WQv Design Event	Annual Runoff Capture
Georgia	30 mm	85%
Maryland	23 mm to 25 mm	90%
Ontario	25 mm	90%
New York	20 to 33 mm	90%
Virginia	25 mm	90%

WATER QUALITY FLOW RATE METHODOLOGY

The WQF is a design flow rate that is selected based on achieving the same objectives as the WQV (treating 80% to 90% of the annual volume of runoff). The analysis of the WQF follows the same level of analysis as that of the WQV by using continuous simulation to analyze historical rainfall records using.

The proposed method to determine WQF is based a continuous simulation model for data analysis. The model is based on the United States Environmental Protection Agency's Stormwater Management Model (SWMM) Version 4.3. To determine the WQF, the RAIN and RUNOFF modules were used for the analysis. This section presents key elements for each module, as adapted from Huber and Dickinson (1988) and James et al. (1999). User input requirement for these SWMM modules include the size of the drainage area, the percentage of imperviousness and local continuous historical rainfall data. Hydrologic assumptions used in the model are listed in Table 2.

Note: The SWMM module was designed using U.S. units of measurement. Therefore references to the equations used in the explanation of the analysis are based on U.S. units.

Table 2. Hydrologic Assumptions for Development of WQF. (Graham et al., 2000)

Parameter	Value
Width (m2)	2 (A) 0.5
Slope	2%
Impervious Depression Storage (mm)	4.86
Pervious Depression Storage (mm)	0.51
Impervious Mannings n	0.015
Pervious Mannings n	0.25
Maximum Infiltration Rate (mm/hr)	62.48
Minimum Infiltration Rate (mm/hr)	9.91
Decay Rate of Infiltration (s^{-1})	0.00055
Infiltration Regeneration (s^{-1})	0.01
Daily Evapotranspiration (mm/day)	0.25
Storm inter event time (hours)	2.0

Rainfall Data

Rainfall data is read by the program in the RAIN module. The program is designed to read 15 minute, daily rainfall data with a 0.25 mm resolution. The resolution, 0.25 mm per tip, of the rain gauge is typical of tipping bucket rain gauges currently used in the field. Rainfall data was formatted to the National Climatic Data Centre (NCDC) format in order to be read by the program. A small sample of the data in NCDC format is shown in Figure 2, with a table defining each column. Each row represents precipitation measurements from one calendar day. For calendar days where no rainfall data were recorded, the program would categorize it as inter event period.

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15M19210702QPCPHI19840600180041430000001   1445000001   2400000003   2500000005
15M19210702QPCPHI19840600190040015000002   0030000002   0045000002   2500000006
15M19210702QPCPHI19840600230071800000004   1815000002   2100000007   2115000011   2130000011   2145000011   2500000046
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15M19210702QPCPHI	**1984**	**06**	**0018**	**004**	**1430**	**00000001^^**	**…**
Identification (17 characters)	Year (4)	Month (2)	Day (4)	Number of time steps recorded (3)	Time of Tip (24 hr) (4)	Number of Tips (10)	continues as required (10)

Figure 2. Sample NCDC Rainfall Format

The program is designed to calculate flow rates in 5 minute time steps. Daily rainfall records, in fifteen minute increments of time, are read by the program and distributed evenly, in 5 minute increments, within a 15 minute period. All rainfall records in the winter periods were treated as rainfall precipitation and not snow.

Rainfall Excess

Rainfall excess is defined as the rainfall intensity minus evaporation and infiltration. Evaporation during rainfall events is presumed to be zero, therefore, providing a conservative estimate of intensity. Infiltration is calculated cumulatively using Horton's Equation (James et al., 1998) as shown in equation [1]. Once the maximum infiltration capacity is reached, the program routes all additional runoff as impervious surface runoff.

[1] $$f_p = f_\infty + (f_o - f_\infty)e^{-\alpha t}$$

where,
f_p = infiltration capacity into soil (ft/s)
f_∞ = minimum infiltration (ft/s)
f_o = maximum infiltration (ft/s)
t = time from beginning of storm (seconds)
α = decay coefficient (s^{-1})

During inter event periods the program runs a regeneration subroutine to determine the availability of infiltration. Regeneration of infiltration is calculated using equation [2].

[2] $$f_p = f_\infty + (f_o - f_\infty)e^{-\alpha_d (t - t_w)}$$

where,
α_d = regeneration coefficient (s^{-1})
t_w = hypothetical projected time at which $f_p = f_o$

Flow Rates

Flow rates are calculated based on the RUNOFF module in 5 minute time steps. The total drainage area is divided into three sub-catchment areas: 1) impervious area with depression storage; 2) impervious area without depression storage; and 3) pervious area with depression storage. Overland flows are calculated for each sub-catchment area by approximating them as non-linear reservoirs (). The outflow from the total drainage area is equal to the sum of all three sub-catchment flow rates.

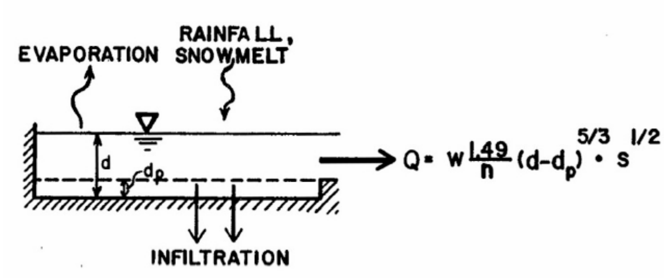

Figure 3. Non-linear reservoir model of subcatchment (Huber and Dickonson, 1988)

Sub-catchment flow rates are determined by combining the Continuity equation [3] and Manning's equation [4] for overland flow. The resulting relationship is identified as a non-linear reservoir equation, equation [5], and is used to determine the water depth at the end of each time step. Once the water depth is determined, it is substituted back into equation [4] to determine the outflow from the sub-catchment area.

[3] $$\frac{dV}{dd} = A\frac{dd}{dt} = A \cdot i * -Q$$

Where,

V = volume of water on the subcatchment (ft^2)
d = water depth (ft)
t = time (seconds)
A = surface area of sub-catchment (ft^2)
i* = rainfall excess, rainfall intensity minus evaporation/infiltration rate (ft/s)
Q = outflow rate (cubic feet per second)

[4] $$Q = W\frac{1.49}{n}(d - d_p)^{5/3} S^{1/2}$$

Where,

W = subcahment width (ft)
n = Manning's roughness coefficient (dimensionless)
d_p = depth of depression storage (ft)

Combining the Continuity equation [3] and Manning's equation [4], the following relationship is determined:

[5] $$\frac{dd}{dt} = i * -\frac{1.49 \cdot W}{A \cdot n} \cdot (d - d_p)^{5/3} S^{1/2} = i * +WCON \cdot (d - d_p)^{5/3}$$

Where width, slope and roughness are combined into one parameter, WCON,

[6] $$WCON = -\frac{1.49 \cdot W \cdot S^{1/2}}{A \cdot n}$$

Equation [7] is calculated at each time step by averaging the net inflow and outflow over the time step.

[7] $$\frac{d_2 - d_1}{\Delta t} = i^* + WCON \cdot (d_1 + 0.5(d_2 - d_1) - d_p)^{5/3}$$

Where,

d_1	=	water depth at the beginning of the time step (ft)
d_2	=	water depth at the end of the time step (ft)
Δt	=	time step, 5 minutes (seconds)

The Newton-Raphson iteration is used to solve for d_2, is substituted back into equation [4] as parameter d to determine the flow rate at the end of time step.

Note: Depression storage is calculated cumulatively and availability is limited depending on the depth at the beginning of the time step. Once the depth of the depression storage has reached its maximum, no further adjustment for depression storage is taken for the remaining time steps. Within the regeneration subroutine of the program, regeneration of depression is calculated using equation [8].

[8] $$\frac{dd_{p2}}{dt} = d_{p1} - SVAP$$

Where,

dp2	=	Water depth in depression storage at end of time step (ft)
dp1	=	Water depth in depression storage at the beginning of the time step (ft)
SVAP	=	Evaporation rate (ft/s)

Flow Distribution Curve

For each time step of calculations, a hydrograph is created for each rainfall event. The total volume of runoff accumulated for the entire period of the rainfall record, based on each time step and by event is determined using equation [6]. The average annual volume of runoff is the total volume, from equation [6], divided by the number of years of rainfall record.

[6] $$V_{Total} = \sum_{n}^{m} \frac{Q_n}{t}$$

where,

V_{Total}	=	Total volume of runoff for the entire period of the rainfall records (ft^3)
n	=	the *n* th time step
m	=	the total number of time steps analyzed, per event
Q	=	flow rate (ft^3/s) at end of time step
t	=	program time steps, 5 minutes (seconds)

Concurrently, a flow distribution curve of annual runoff capture by increments of flow rate is created. The total number of increments of flow rate is 30 and is calculated by the sequence shown in equation [7] and in units of litres per second. For each increment of flow rate, the cumulative volume of runoff that is equal to or less than the increment of the flow rate is calculated. For time steps where the event flow rate exceeds the incremental flow rate, the volume is calculated using the incremental flow rate only. The remaining volume is assumed to have by-passed. This cumulative volume of runoff is called the volume of runoff captured. The annual volume of runoff captured is the volume of runoff captured divided by the number of years of the rainfall data used. For the flow distribution curve, the annual runoff captured is expressed as a percentage of the total annual volume of runoff.

[7] $$Q_n = n^2$$

Where,

Q_n	=	Flow rate (L/s)
n	=	An integer from 1 to 30 in increments of 1

The WQF may then be selected from the flow distribution curve by selecting the flow rate that contributes to 80% to 90% of the annual runoff volume.

RESULTS AND DISCUSSION

Sample Analysis

This section provides a sample analysis to determine the WQF for the City of Toronto, Ontario. The purpose is to demonstrate how WQF is interpreted and applied. Rainfall data from 1982 to 1999 and recorded at daily 15 minute intervals was obtained for the City of Toronto. The drainage area selected in the analysis was one hectare. The site was assumed to be 100% impervious to provide a conservative estimate of flow rates and volume estimates. The results of the flow distribution curve for the City of Toronto is tabulated in Table 3 and plotted in Figure 4. The WQF may then be determined from the flow distribution curve.

Table 3. Flow Distribution Table by Cumulative Annual Volume of Runoff for a 1 Hectare, 100% Impervious Catchment, City of Toronto Rainfall Set, 1982 to 1999.

Treatment Flow Rate m^3/s	Ave. Annual Volume of Runoff Less than the Flow Rate m^3	Ave. Annual Volume of Runoff Greater than the Flow Rate m^3	Total Ave. Annual Volume of Runoff m^3	Cumulative Annual Runoff Treated %
0.001	23223	95716	118933	19.5
0.004	60308	58632	118933	50.7
0.009	86243	32702	118933	72.5
0.016	99782	19153	118933	83.9
0.025	106793	12143	118933	89.8
0.036	110874	8060	118933	93.2
0.049	113463	5471	118933	95.4
0.064	115231	3703	118933	96.9
0.081	116418	2516	118933	97.9
0.100	117223	1710	118933	98.6
0.121	117782	1152	118933	99.0
0.144	118162	772	118933	99.4
0.169	118400	533	118933	99.6
0.196	118494	439	118933	99.6

If the WQV objective is to treat 90% of the annual runoff volume, then similarly, the runoff flow rate that contributes to this annual runoff volume can be determined by corresponding the values shown in Figure 4. From Figure 4, the runoff rates that contributes to 90% of the annual runoff volume is 0.036 m^3/s or less. As such, the flow based technology can be designed to treat up to a maximum flow rate of 0.036 m^3/s to ensure 90% of the annual volume of runoff is treated. Similarly, if the WQV objective is 85% treatment of the annual runoff volume, the WQF is 0.018 m^3/s.

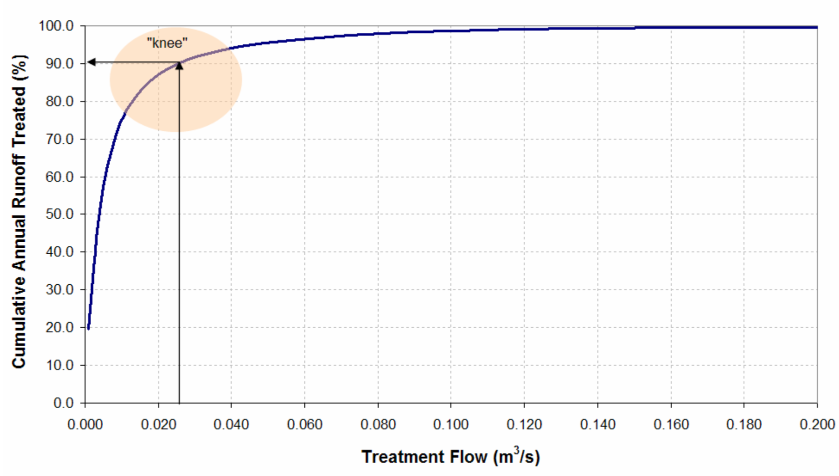

Figure 4. Flow Distribution Curve by Cumulative Annual Volume of Runoff for a 1 Hectare, 100% Impervious Catchment, City of Toronto Rainfall Set, 1982 to 1999.

The plot of the flow distribution curve, Figure 4, illustrates that for incremental changes at the lower range of flow rates a large incremental change in volume occurs. This is illustrated by a steep rising slope at the lower flow rates (up to 0.015 m^3/s). This large incremental change in volume is due to the fact that the majority of the rainfall events consist of small storms and contributes to a greater percentage of the annual volume of runoff due to frequency of occurrence. Conversely, for the larger flow rates, the slope of the curve levels out and the increase in volume captured is marginal. This illustrates that large storm events occur less frequently and the volume of runoff contributed do not make up the majority of the annual runoff volume.

The inflection point or "knee" of the graph is typically found at the 80^{th} to the 90^{th} percentile of the annual runoff volume. The rate of return on the volume of runoff treated rapidly diminishes after the "knee". The cost to build a treatment facility to treat a higher flow rate or runoff volume is much greater than the return on removal efficiency since large storms occur infrequently. In addition, the gain in water quality benefits would be less as the additional gain in volume treated is minimal when compared to the total annual volume of runoff.

Limitations

The use of daily rainfall data collected at larger intervals of time, e.g. record collected every 60 minute versus every 15 minute, will yield more conservative estimates. In the RAIN module of the simulation program, hourly readings are distributed into the first 15 minutes of the hour. Therefore 60 minute daily rainfall data is assumed to fall in the first 15 minutes of each hour rather than over the entire hour. This results in more conservative estimates of runoff rates and volumes of runoff.

CONCLUSIONS AND RECOMMENDATIONS

A methodology for the determination of a WQF by analyzing local historical 15 minute, daily rainfall records has been presented using a continuous hydrologic simulation model based on the USPEA SWMM Version 4.3. The results conclude that the WQF may be selected to achieve the same objective as the WQV, treatment of 80% to 90% of the annual volume of runoff. The WQF may be applied by regulators and designers as a minimum standard for the sizing of flow based water quality treatment systems.

The WQF may be calculated for other geographic locations in North America using continuous 15 minute daily historical rainfall data. The model is sensitive to 60 minute rainfall data and over estimates the runoff rates and runoff volumes. It is recommended that any analysis of WQF be completed based on continuous 15 minute, daily rainfall records.

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