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A Modified Rational Method Approach for Calculating First Flush Design Flow Rates to Mitigate Nonpoint Source Pollution from Stormwater Runoff

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Abstract: Mitigating nonpoint source pollution from stormwater runoff demands effective strategies for treating the first flush depth. Whether through off-stream storage or pass-through treatment devices, designing diversion structures and filtering materials is critical. This study proposes a streamlined procedure for determining first flush design flow rates, employing the modified rational method and rainfall intensity–duration equations applicable to any U.S. location. The dimensionless solution, which is presented as an equation requiring an iterative calculation for the desired flow rates, is complemented by precision graphs. Examples from the semi-arid Southwestern United States illustrate the methodology's utility.

Keywords: stormwater runoff; water quality; pollution; first flush; modified rational method; runoff separation; runoff filtration

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1. Introduction

The "first flush" is a concept familiar to designers of stormwater treatment systems and refers to the initial surface runoff of a rainstorm [1–3]. During this phase, water pollution entering storm drains in areas with high proportions of impervious surfaces is typically more concentrated than that from the rest of the storm. Experimental evidence underscores the substantial pollutant loads associated with initial stormwater wash-off, particularly in catchments dominated by impervious surfaces and rapid runoff conveyance [4]. Regulatory measures often target the treatment of the first flush depth, emphasizing detention or filtration [5,6]. While definitions of a first flush event vary, its significance in pollutant transport is widely acknowledged [2,7–11].

The occurrence and the need to treat the initial cleansing or the first flush effect of contaminants from urban catchments was recognized more than a century ago by Metcalf and Eddy [12]. The urgency to remedy these high concentrations of stormwater pollutants continues worldwide as urbanization intensifies (see, for example [13–15]). It is impractical to treat all the runoff from a large storm. However, pass-through filtering and off-stream storage of the first flush from these storms, as well as the entire runoff volume of smaller, more frequent storms, can reduce the long-term nonpoint pollutant loads from stormwater significantly [16].

Treating the first flush depth of runoff, either by storing it until it can be treated and released or passing it through a filtering device that discharges it immediately into the downstream drainage system, is one way of mitigating nonpoint source pollution from stormwater. A diversion structure must be installed that redirects runoff until the desired capture volume fills if off-stream storage is used. The filtering material must be specified if a pass-through treatment device is employed. In either case, a flow rate corresponding to the first flush runoff depth must be determined.

This study presents a straightforward procedure for calculating first flush design flow rates based on the modified rational method and rainfall intensity–duration equations that can be easily determined for any location in the United States [17]. However, the approach can be applied anywhere rainfall data are available to fit an intensity–duration equation like that used in this study. The solution is made dimensionless by grouping parameters and takes the form of an equation that must be solved iteratively for the rainfall duration that produces the desired flow rate. However, graphs that provide solutions of sufficient precision can be created quickly. Examples of such graphs are presented for the semi-arid Southwestern United States, where a single set of rainfall intensity–duration coefficients applies to all the average recurrence intervals.

2. Treating the First Flush of Stormwater Runoff

Experimental studies in catchments where a substantial portion of the land is covered by impervious surfaces and where artificial drainage channels quickly transport runoff to the outlet confirm the comparatively large pollutant loads from the initial wash-off [4,14]. For this reason, regulations for mitigating nonpoint source pollution from small catchments often require treating the first flush depth of the runoff (that is, an initial amount of runoff from a catchment) either by detaining the stormwater until it can be treated and released [5] or by passing it through a filtering device [6]. Saget et al. [7,8] consider the first flush when 80% of the pollutant mass is transported in the initial 30% of the runoff volume, a definition also adopted by Bertrand-Krajewski et al. (1998) [9]. However, other definitions of a first flush runoff event have been proposed [2,10,11].

When first flush runoff is held in a storage area that is not connected directly to a drainage channel (an off-stream storage area), it must be diverted from the contributing stream until the desired volume is captured [18,19]. Where topographic and other conditions permit it, horizontal weirs, whose crests are at about the same elevation as the maximum water level in an adjacent detention area, are often used to redirect first flush runoff from the main channel and allow most of the following flow to bypass the off-stream storage. If such a diversion method is not feasible, controls like the Contech Engineered Solutions StormGate® (Contech 2018) [20] are designed to send initial runoff into storage until the first flush design flow is reached. Most discharge above this set value then avoids being channeled off-line.

Pass-through treatment devices typically house rechargeable, media-filled cartridges that trap particulates and adsorb pollutants such as total suspended solids, hydrocarbons, nutrients, and metals. The number of filter media must be determined based on the first flush design discharge to provide sufficient treatment capacity [21,22]. For example, to size Contech's StormFilter® [23,24], the design flow rate is divided through the StormFilter's cartridge treatment flow rate to determine the number of cartridges required. Depending on the targeted pollutants, the cartridge flow rate may vary from 0.125 to 1.0 L per second (2 to 15 gallons per minute). The structure housing the filters can then be sized to accommodate the required number of cartridges.

Ahlfeld and Minihane (2004) [25] developed a probabilistic method to find the first flush design discharge using the rational method to relate flow and rainfall intensity. However, most often, a straightforward approach is taken by summing the accumulated runoff obtained from a calculated hydrograph, in which flow rates increase monotonically until the peak flow rate is reached, to establish the time and the discharge at which the required runoff volume has accumulated [21,26,27]. Following this approach, Froehlich (2009a) [28] developed an uncomplicated graphical procedure for calculating the first flush flow rates from small catchments based on Natural Resources Conservation Service (NRCS) hydrologic methods.

Building on the rationale of Froehlich (2009a) [28], a method is presented for calculating the first flush design flow rate based on runoff hydrographs developed from an extension of the rational formula using a technique known as the modified rational method or MRM [29–33]. Applied with short-duration rainfall intensity–duration equations whose parameters can be obtained without difficulty for most of the United States [34], the MRM provides closed-form expressions for flow rates corresponding to specified first flush runoff volumes. Dimensionless parameters streamline the analysis, thereby providing a comprehensive and practical tool for stormwater management.

3. MRM Hydrology

The rational formula or rational method [35], which is used widely to determine the needed flow capacities of minor drainage structures, gives the peak discharge of stormwater runoff from a catchment as

$$Q_p = k_u C A_c \times \bar{i}(t_d) \tag{1}$$

where C = the runoff coefficient representing the fraction of the incident precipitation that appears as surface runoff, A_c = the contributing catchment area, $\bar{i}(t_d)$ = the average rainfall intensity for a storm of duration t_d with depth units of rainfall per hour, and k_u = a conversion factor that depends on the units of Q_p, A_c, and $\bar{i}$ (for example, $k_u = 0.02778$ for $Q_p = \text{m}^3/\text{s}$, A_c = ha, and $\bar{i}$ = cm/h; or $k_u = 1.008$ for $Q_p = \text{ft}^3/\text{sec}$, A_c = ac, and $\bar{i}$ = in/h). Theoretically, C can range from nearly zero to one depending on land use, cover conditions, soil characteristics, watershed slope, and rainfall intensity. However, in practice $0.05 \leq C \leq 0.95$, where the upper limit represents highly impervious surfaces such as roofs and pavements. When used for a design based on annual exceedance probability, Q_p is considered to possess the same average annual exceedance probability as $\bar{i}$, and C expresses the rate of proportionality between $\bar{i}$ and Q_p.

Because average rainfall intensity decreases as rainfall duration increases, Q_p, which is given in Equation (1), usually reaches its maximum value when $t_d = t_c$ where t_c = the time-of-concentration of the catchment. In an ideal sense, t_c is the time needed for water to flow from the most remote point of a catchment to its outlet, where remoteness relates to the time of travel rather than the length of the flow path. However, situations may exist where a portion of a catchment drains to the outlet much faster than the rest in which case the peak discharge may occur when $t_d < t_c$ and only part of the watershed contributes to the outflow. Although such circumstances may not be rare, the limited size of the catchments considered in this analysis precludes the frequent occurrence of this condition.

The original idea behind the rational method is that if rainfall of intensity $\bar{i}$ begins instantaneously and continues indefinitely, the rate of runoff increases until the time of concentration when all of the watershed contributes to the flow at the outlet [36]. When viewed this way, the rational formula is a process-based model of catchment runoff in which C accounts for all rainfall losses (interception, depression storage, and infiltration). However, the relation can also be regarded as a statistical correlation between Q_p and the product of the independent variables $\bar{i}$ and A_c where C plays the role of a proportionality coefficient [37–39]. No matter the interpretation, the rational formula has been used in the United States for over a century [40] and continues to be applied worldwide for designing minor drainage structures [41].

Perhaps the most favorable aspects of the rational formula are that it is comparatively easy to apply, rainfall intensity–duration relations are usually available, and the information needed to evaluate the catchment time-of-concentration and the runoff coefficient can be obtained without difficulty [42]. Despite several deficiencies and limitations (see American Public Works Association (APWA), 1981; Walesh, 1989; and Westphal, 2001, for thorough assessments) [33,43,44], the rational formula is suitable for calculating the stormwater runoff from small catchments, particularly in urban areas where a large percentage of the land surface is impervious.

The MRM, an American Society of Civil Engineers (ASCE) standard practice for designing urban stormwater systems ([45], Section 4.1.8), which also is applied worldwide [46,47], relies on the same assumptions as those of the rational formula and the notion that the runoff coefficient C is constant with respect to time and rainfall intensity during a storm. For $t_d = t_c$, runoff hydrographs are constructed by considering discharge to increase at a linear rate from the start of rainfall over a period t_c and then decrease at a linear rate over a time t_c. When $t_d > t_c$, the entire catchment area A contributes to the flow at the outlet, and discharge remains constant at the peak rate Q_p for the period. The discharge then decreases at a linear rate over t_c. When $t_d < t_c$, the flow increases linearly to reach Q_p at time t_d and then remains constant until $t = t_c$. The flow rate then drops at a linear rate over a period t_d. The total duration of runoff equals $t_c + t_d$ under all circumstances. Finally, based on MRM reasoning, the fraction of the catchment draining to the outlet when $t_d < t_c$ equals the ratio t_d/t_c, which gives

$$A_c = \begin{cases} A, \text{ for } t_d \geq t_c \\ A\frac{t_d}{t_c}, \text{ for } t_d < t_c \end{cases} \tag{2}$$

To illustrate the idea, several MRM runoff hydrographs developed for a small catchment for which t_c = 30 min are shown in Figure 1, where dimensionless discharge $Q_* = Q/Q'_p$ is graphed against dimensionless time $t_* = t/t_c$ for several values of t_d, Q'_p = the peak discharge from the catchment for $t_d = t_c$, and t = the time since the start of rainfall. All the runoff hydrographs are equally likely to occur based on MRM reasoning. Dimensionless peak discharge $Q_{p*} = Q_p/Q'_p$ produced by runoff hydrographs from a catchment for several t_c values is graphed against $t_{d*} = t_d/t_c$ in Figure 2. The relations form continuous curves that reach a maximum when $t_{d*} = 1$ (that is, when $t_d = t_c$). However, the rising portion of the dimensionless hydrograph for $t_{d*} \leq 1$ is of particular interest to the following analysis.

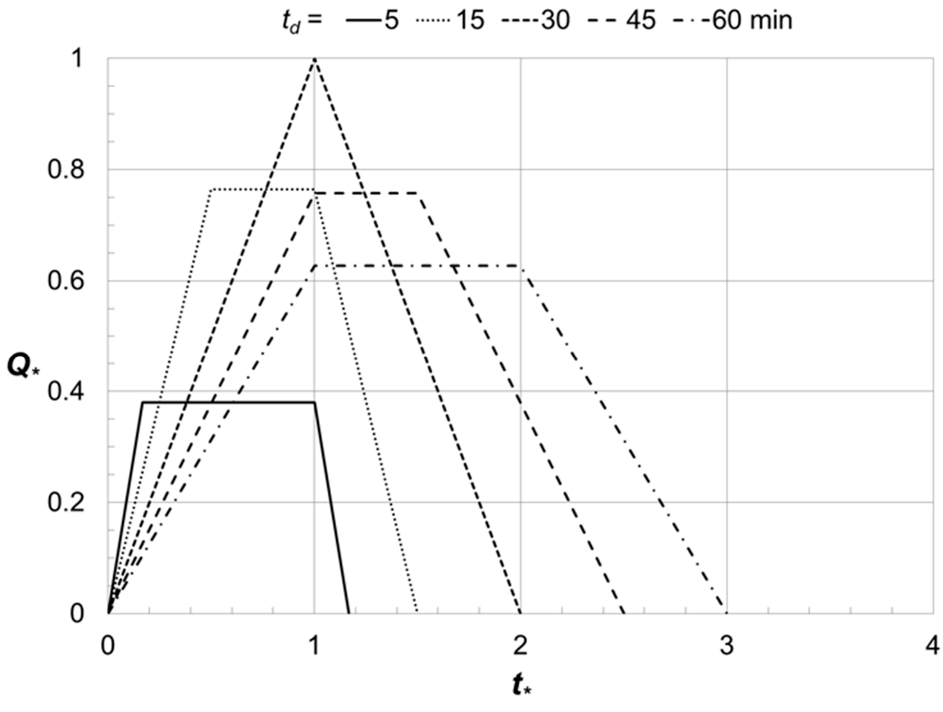

Figure 1. Runoff hydrographs from a small catchment given through the modified rational method show the effect of storm duration on peak flow rates and hydrograph shape.

The maximum size of catchments to which the MRM can be applied is often limited. For example, the *Iowa Statewide Urban Design and Specifications Manual* (Iowa Center for Transportation Research and Education, 2007, page 2C-9-6) [48] and the *Virginia Stormwater Management Handbook* (Virginia Department of Conservation and Recreation, 1999, page 4–17) [49] restrict the use of the MRM to drainage areas of 8 ha (20 ac) or less, while the *Georgia Stormwater Management Manual* (Atlanta Regional Commission, 2001, page 2.1-8) [50] suggests an upper limit of 2 ha (5 ac). Chow et al. [32] recommend the application of the MRM to catchments with areas no larger than 12 ha (30 ac). Based on these customary practices, the writer concludes that the MRM applies best to *small* catchment draining areas of 12 ha (30 ac) or less.

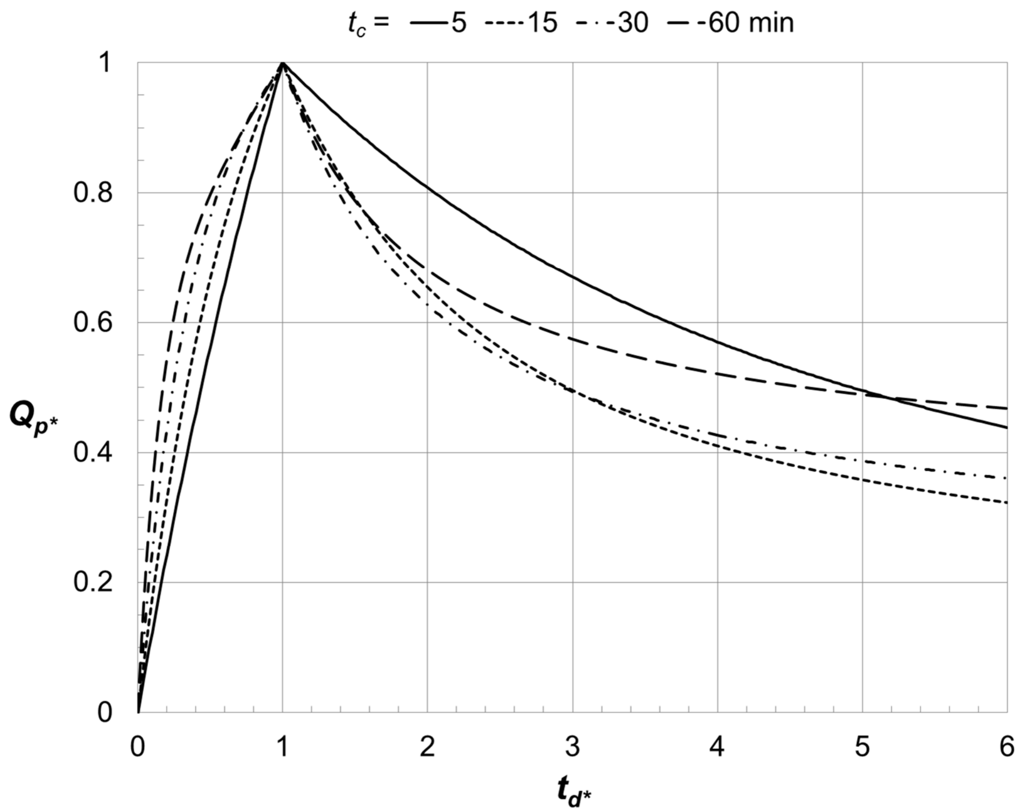

Figure 2. Normalized peak runoff rate as a function of normalized storm duration for several values of time of concentration.

4. Rainfall Intensity–Duration Relation

Average rainfall intensity for a specified storm duration t_d is obtained from an equation of the following form:

$$\bar{i}(t_d) = \left(\frac{i_p - i_o}{mt_d}\right)\left(1 - e^{-mt_d}\right) + i_o\,, \tag{3}$$

where i_p = the peak rainfall intensity (depth units per hour), i_o = a constant rainfall intensity (depth units per hour), m = a coefficient that describes the nonuniformity of rainfall intensity during the storm (units of h^{-1}), and t_d = the rainfall duration (hours). Froehlich (2010) [34] presents an uncomplicated procedure for determining the coefficients in Equation (3) for locations in the United States covered by NOAA Atlas 14, Volumes 1, 2, 3, and 4 [51–54]. The coefficients were developed from rainfall data for durations ranging from five minutes to one hour. For this reason, the formula should be used for storm durations of no longer than one hour.

For the semi-arid Southwestern United States covered by NOAA Atlas 14, Volume 1 (Arizona, Southeast California, Nevada, New Mexico, and Utah), $i_p = i_{p*} \times \bar{i}_{60}$, $i_o = i_{o*} \times \bar{i}_{60}$, and $m = m_*/1h$ where $i_{p*} = 4.639$, $i_{o*} = 0.362$, $m_* = 6.676$, and $\bar{i}_{60}$ = the average 60 min rainfall intensity in rainfall depth per hour [34]. The coefficients apply to all the average recurrence interval (ARI) rainfall in the region.

5. Calculating the First Flush Design Flow Rate

The amount of contaminants in the total annual runoff from a catchment removed by treating the first flush varies depending on the treatment practice's effectiveness and the geographical region's rainfall characteristics. Because comparatively small, frequently occurring storms account for most of the rainfall that generates stormwater runoff, the same storms also account for most annual pollutant loadings. For this reason, reducing harmful water quality impacts is possible by managing all runoff from frequently occurring small storms and a portion of the runoff from more significant events [55,56].

The quantity of the initial runoff that needs to be treated, often called the water quality volume, is usually defined as a depth of rainfall excess draining from the catchment, which is denoted here by D_f and referred to as the water quality or first flush capture depth [57]. The volume of first flush runoff that needs to be stored or filtered is $V_f = D_f \times A_c$. Regulations specify D_f directly, often 6.35 or 12.7 mm (0.25 or 0.50 in), or they require that D_f equals the total runoff from a storm with a specified rainfall depth or ARI.

The flow rate Q_f corresponding to a specified first flush runoff volume is obtained in the following analysis from appropriate MRM hydrographs such as those shown in Figure 1. As explained, if a specified first flush volume fills before the hydrograph peak discharge is reached, the volume corresponds to a unique flow rate used to design a control measure. However, suppose the peak discharge occurs before capturing the first flush runoff volume. In that case, the diversion structure or filtering device must be sized based on the maximum peak discharge from the catchment, which occurs when $t_d = t_c$. One can determine quickly if the specified capture volume fills before the peak discharge is reached by comparing it to the runoff volume $V' = \frac{1}{2}Q'_p t_c$ under the rising side of the triangular hydrograph produced when $t_d = t_c$. If $V_f > V'$ then $Q_f = Q'_p$; otherwise, $Q_f < Q'_p$ and does not need to be determined based on MRM runoff hydrographs for which $t_d < t_c$.

The capture volume is normalized to simplify the calculation of the first flush design discharge as follows:

$$V_{f*} = V_f / V' \tag{4}$$

If $V_{f*} \geq 1$, the specified volume does not fill until $t > t_c$ and the control devices must be designed for a flow rate equal to Q'_p. When $V_{f*} < 1$, the appropriate hydrograph $t_d < t_c$ that generates a runoff volume of V_f at its peak (that is, when $t = t_d$) must be determined to obtain the corresponding value of Q_{p*}, which defines the normalized first flush design flow rate $Q_{f*} = Q_f / Q'_p$.

For conditions where $V_{f*} < 1$, V_{f*} is equated to the volume under the rising side of the MRM hydrograph as follows:

$$V_f = \frac{1}{2} Q_f t_{df} \tag{5}$$

where t_{df} = the storm duration producing the first flush capture volume. Dividing through Equation (5) using V' provides the following expression:

$$V_{f*} = Q_{p*} t_{df*} \tag{6}$$

where $t_{df*} = t_{df}/t_c$. With Qp given in Equation (1), A_c by Equation (2), and $\bar{i}(t_d)$ by Equation (3):

$$V_{f*} = \left\{ \frac{\left(\frac{i_{p*} - i_{o*}}{m_* t_c t_{df*}}\right)\left[1 - \exp\left(-m_* t_c t_{df*}\right)\right] + i_{o*}}{\left(\frac{i_{p*} - i_{o*}}{m_* t_c}\right)[1 - \exp(-m_* t_c)] + i_{o*}} \right\} \times t_{df*}^2 \tag{7}$$

where the runoff coefficient C is considered constant. The expression given in Equation (7) can be solved iteratively for t_{df*} in terms of V_{f*} and the other specified parameters (that is, i_{p*}, i_{o*}, m_*, and t_c). The value of t_{df*} is then used in Equation (6) to recover Q_{f*}. The required calculations can be carried out rapidly using commonly available spreadsheet software.

Graphical solutions can also be prepared to provide rapid assessments of Q_{f*} and t_{df*} of sufficient precision for designing first flush runoff controls. For example, the graphs shown in Figures 3 and 4 were created using the coefficients i_{p*}, i_{o*}, and m_* given previously for the semi-arid Southwestern United States covered by NOAA Atlas 14, Volume 1 [51]. The graphs give Q_{f*} and t_{df*} for any ARI rainfall within the region.

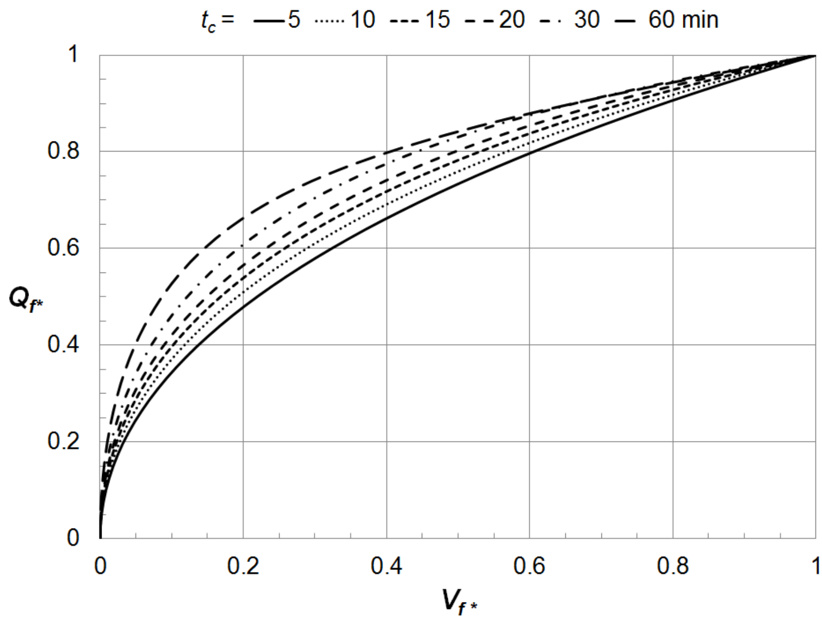

Figure 3. Normalized first flush design flow rate as a function of normalized first flush capture volume for several values of time of concentration for the semi-arid Southwestern United States covered by NOAA Atlas 14, Volume 1.

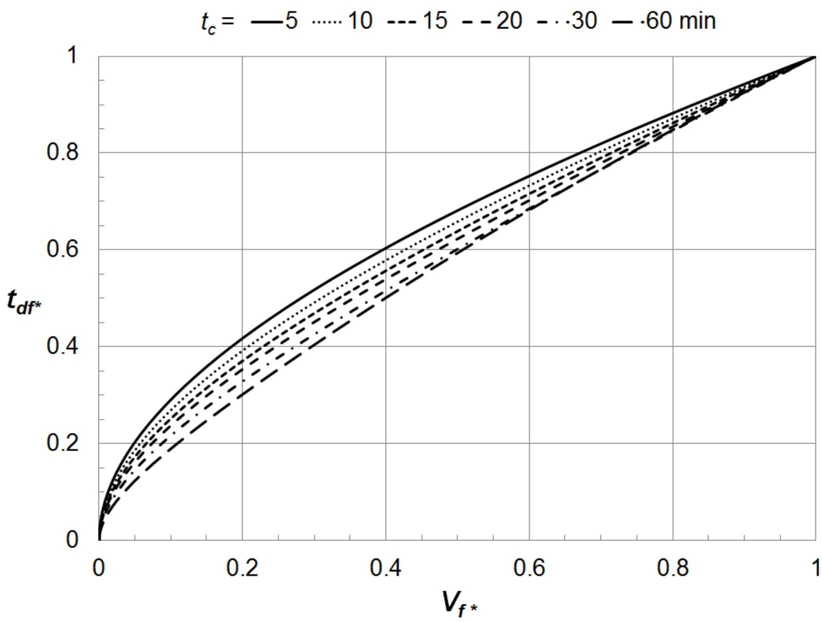

Figure 4. Normalized first flush storm duration as a function of normalized first flush capture volume for several values of time of concentration for the semi-arid Southwestern United States covered by NOAA Atlas 14, Volume 1.

Graphical relations for Q_{f*} presented in Figure 3 are approximated closely through numerical expressions of the form

$$Q_{f*} = V_{f*}^{(a+bV_{f*}^{c})} \tag{8}$$

where a, b, c = coefficients that are functions of t_c. Optimal values of a, b, and c for t_c ranging from 1 min to 60 min are given in Table 1 and are plotted in Figure 5. Relations for a, b, and c shown in Figure 5 are matched closely through the following expressions:

$$a = 18.7t_c^4 - 22.4t_c^3 + 8.43t_c^2 - 1.11t_c + 0.519 \ (\mathrm{r}^2 = 0.996) \tag{9}$$

$$b = -19.3t_c^4 + 23.5t_c^3 - 8.35t_c^2 + 0.358t_c + 0.0245 \ (\mathrm{r}^2 = 0.996) \tag{10}$$

$$c = 1.01t_c^4 - 0.785t_c^3 - 0.969t_c^2 + 0.461t_c + 0.290 \ (\mathrm{r}^2 = 0.999) \tag{11}$$

where t_c is in hours and coefficients of determination (r^2), which are based on fits to the tabulated data, are given in the right-hand sides of Equations (9)–(11). The expression for Q_{f*} given in Equation (8) can be used most effectively in spreadsheet calculations to avoid iterative solutions. Graphical relations for Q_{f*} and t_{df*}, like those presented in Figures 3 and 4, and relations similar to Equations (8)–(11) can be developed for other regions of the United States for which rainfall intensity–duration equation coefficients i_{p*}, i_{o*}, and m_* are given by Froehlich (2010) [34].

Table 1. Coefficients [a] a, b, and c for values of t_c ranging from 1 min to 60 min.

t_c (min)	t_c (h)	Coefficients		
		a	b	c
1	0.0167	0.4955	−0.0205	0.2899
2	0.0333	0.4951	−0.0207	0.3030
5	0.0833	0.4878	−0.0515	0.3240
10	0.1667	0.4762	−0.1000	0.3377
15	0.2500	0.4673	−0.1436	0.3351
20	0.3333	0.4627	−0.1813	0.3170
25	0.4167	0.4643	−0.2141	0.2855
30	0.5000	0.4750	−0.2454	0.2435
35	0.5833	0.5010	−0.2832	0.1934
40	0.6667	0.5595	−0.3468	0.1377
45	0.7500	0.7277	−0.5150	0.0776
50	0.8333	1.4305	−1.2156	0.0275
55	0.9167	2.6341	−2.4175	0.0124
60	1.0000	3.9862	−3.7667	0.0072

[a] Coefficients apply only in the semi-arid Southwestern United States where short-duration rainfall intensity is provided in NOAA Atlas 14, Volume 1 (Arizona, Southeast California, Nevada, New Mexico, and Utah).

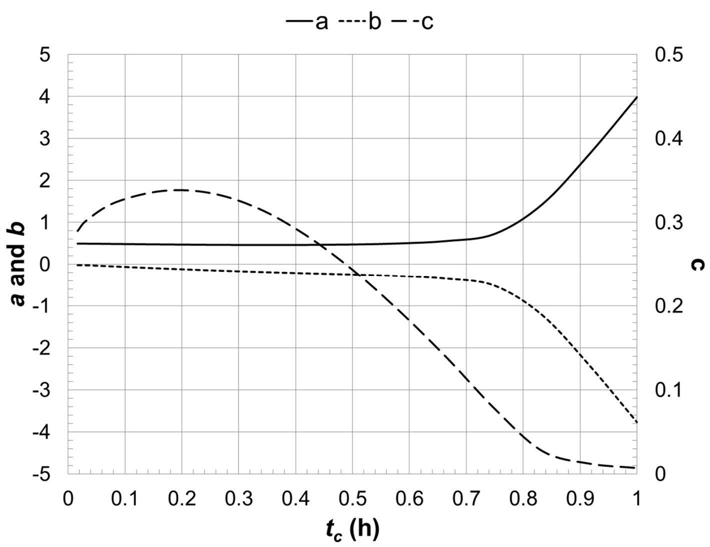

Figure 5. Coefficients a, b, and c as a function of t_c.

6. Application Procedure

Steps taken to apply the procedure developed in this paper to calculate the peak discharge that corresponds to a specified first flush capture volume from a small catchment are as follows:

Step 1. With specified values of $C, A, t_c, i_{p*}, i_{o*}, m_*, \bar{i}_{60}$, and V_f, calculate Q'_p using Equations (1)–(3) and then V'.

Step 2. Calculate V_{f*} from Equation (4).

Step 3. If $V_{f*} \geq 1$, set $Q_f = Q'_p$,; otherwise, proceed to Step 4.

Step 4. Find t_{df*} through an iterative solution of Equation (7) and then obtain Q_{f*} from Equation (6), or, if the catchment is in the Southwestern United States, obtain Q_{f*} directly from Figure 3 or Equation (8) with coefficients a, b, and c given in Equations (9)–(11). Obtain t_{df*} from Figure 4 if desired.

Step 5. Calculate $Q_f = Q_{f*} \times Q'_p$ and the corresponding rainfall duration $t_{df} = t_{df*} \times t_c$.

7. Example Applications

Two examples illustrate applying the procedure described in this paper to calculate first flush flow rates from small catchments in the United States. The first application is to a small multi-family residential development in the country's southeastern region, and the second example describes the calculation of the first flush design flow for an industrial site located in the semi-arid southwest.

7.1. Example A

A first flush runoff depth D_f = 6.35 mm (0.25 in) resulting from a 10-year ARI rainfall on a small multi-family development project in Raleigh, North Carolina, a region of the United States covered by NOAA Atlas 14, Volume 2 [51], is to be treated through a stormwater filtering device before discharge into the primary storm drain system. Parameters that apply to the site are as follows: C = 0.79, A = 2.43 ha, t_c = 12 min = 0.200 h = 720 s, $i_{p*} = 3.605, i_{o*} = 0.405, m_* = 5.342$, and $\bar{i}_{60} = 57$ mm/h (for 10-year ARI rainfall). The rainfall intensity–duration equation parameters i_{p*}, i_{o*}, and m_* only apply to 10-year ARI storms.

The solution for the first flush design discharge is obtained from Equations (6) and (7) as follows:

Step 1. With $t_d = t_c = 0.200$ h, $i_p = i_{p*} \times \bar{i}_{60} = 3.605 \times 57 = 205.5$ mm/h, $i_o = i_{o*} \times \bar{i}_{60} = 0.405 \times 57 = 23.1$ mm/h, $m = m_*/1\text{h} = 5.342\ \text{h}^{-1}$, Equation (3) gives $\bar{i}(t_c) = \left(\frac{205.5-23.1}{5.342\times0.200}\right)\left(1 - e^{-5.342\times0.200}\right) + 23.1 = 135.2$ mm/h. With $A_c = A = 2.43$ ha, $Q'_p = 0.00278 \times 0.79 \times 2.43 \times 135.2 = 0.721\ \text{m}^3/\text{s}$ from Equation (1), which gives $V' = \frac{1}{2} \times 0.721 \times 720 = 259.6\ \text{m}^3$.

Step 2. From Equation (4), $V_{f*} = \frac{\left(\frac{6.35}{1000}\right)\times2.43\times10{,}000}{259.6} = 0.594$.

Step 3. Because $V_{f*} < 1$, proceed to Step 4.

Step 4. The iterative solution of Equation (7) gives $t_{df*} = 0.733$, which gives $Q_{f*} = 0.811$ from Equation (6).

Step 5. The first flush design discharge $Q_f = Q_{f*} \times Q'_p = 0.811 \times 0.721 = 0.585\ \text{m}^3/\text{s}$ and the corresponding rainfall duration $t_{df} = t_{df*} \times t_c = 0.733 \times 12 = 8.80$ min.

Graphs for Q_{f*} and t_{df*} for this particular site where t_c = 12 min and the first flush design flow rate that is based on 10-year ARI rainfall are shown in Figures 6 and 7, respectively.

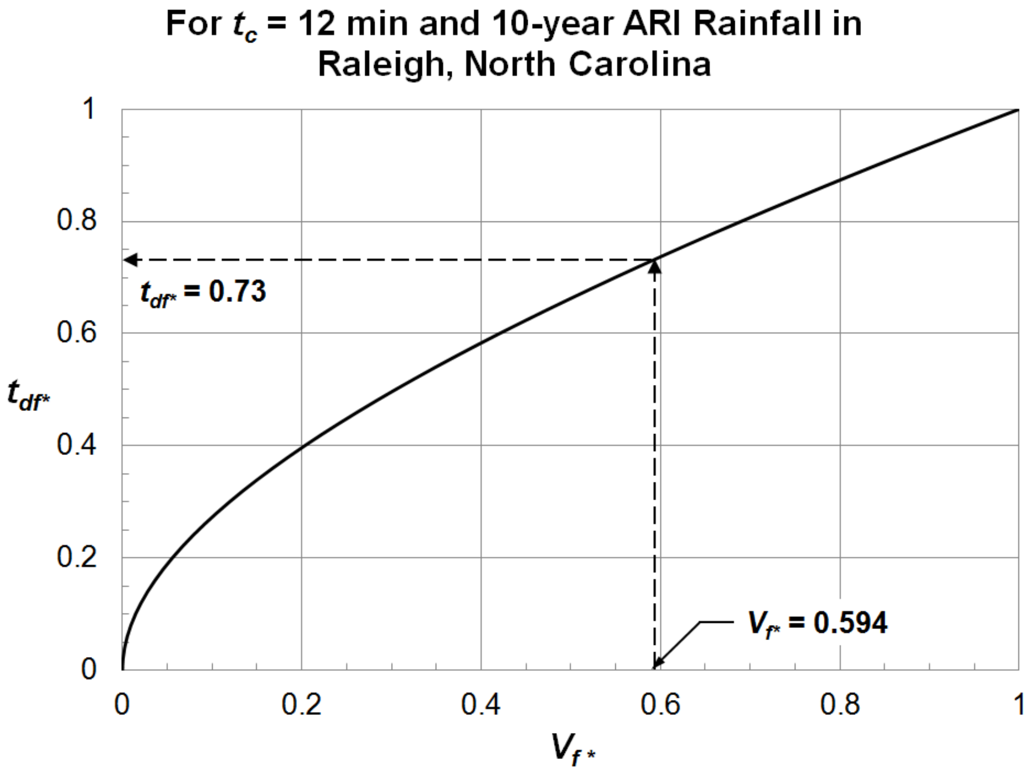

Figure 6. Graphical solution of t_{df^*} for a catchment with t_c = 12 min in Raleigh, North Carolina.

7.2. Example B

The flow rate corresponding to a specified first flush runoff depth of 6.35 mm (0.25 in) from a 10-year ARI storm is found for a small industrial site in Albuquerque, New Mexico, in the semi-arid Southwestern United States. Parameters that apply to the catchment are as follows: C = 0.92, A = 3.24 ha, t_c = 15 min = 0.25 h = 900 s, i_{p^*} = 4.639, i_{o^*} = 0.362, m_* = 6.676, and $\bar{i}_{60} = 29$ mm/h. With A in hectares and $\bar{i}$ in mm/h, the unit conversion coefficient is k_u = 0.00278. The following solution for the first flush design discharge uses graphs in Figures 3 and 4, which apply to all ARI rainfall in the arid Southwestern United States covered by NOAA Atlas 14, Volume 1 [51].

Step 1. With $t_d = t_c = 0.250$ h, $i_p = i_{p*} \times \bar{i}_{60} = 4.639 \times 29 = 134.5$ mm/h, $i_o = i_{o*} \times \bar{i}_{60} = 0.362 \times 29 = 10.5$ mm/h, and $m = m_*/1\text{h} = 6.676\ \text{h}^{-1}$, Equation (3) then gives $\bar{i}(t_c) = \left(\frac{134.5-10.5}{6.676\times0.250}\right)\left(1-e^{-6.676\times0.250}\right) + 10.5 = 70.8$ mm/h. With $A_c = A$ = 3.24 ha, $Q'_p = 0.00278 \times 0.92 \times 3.24 \times 70.8 = 0.587\ \text{m}^3/\text{s}$ from Equation (1), which gives $V' = \frac{1}{2} \times 0.587 \times 900 = 264.2\ \text{m}^3$.

Step 2. From Equation (4), $V_{f*} = \frac{\left(\frac{6.35}{1000}\right)\times3.24\times10{,}000}{264.2} = 0.779$.

Step 3. Because $V_{f*} < 1$, proceed to Step 4.

Step 4. Because the catchment is in the semi-arid Southwestern United States, Q_{f^*} can be found directly from Figure 3 or Equation (8) with coefficients a, b, and c given in Equations (9)–(11). From Figure 3, Q_{f^*} = 0.92. The corresponding value of t_{f^*} obtained from Figure 4 is 0.85.

Step 5. The first flush design discharge $Q_f = Q_{f*} \times Q'_p = 0.92 \times 0.587 = 0.54\ \text{m}^3/\text{s}$ and the corresponding storm duration $t_{df} = t_{df*} \times t_c = 0.85 \times 15 = 12.8$ min.

More precise solutions were found by solving Equations (6) and (7) giving t_{df} = 12.7 min and Q_f = 0.540 m^3/s.

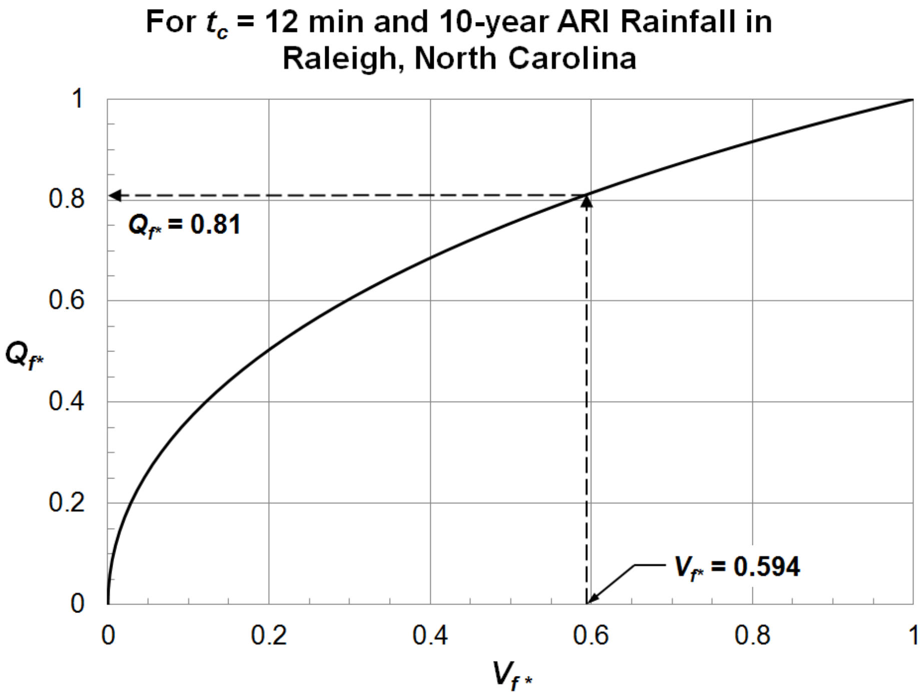

Figure 7. Graphical solution of Q_{f*} for a catchment with t_c = 12 min in Raleigh, North Carolina.

8. Summary and Conclusions

Stormwater first flush runoff held in a storage area not connected directly to a drainage channel (an off-stream storage area) must be diverted from the main channel streamflow until the desired volume is captured. Contaminants, mostly floating debris and suspended solids may also be removed from the first runoff by directing the stormwater through a treatment device. In either case, the structural measure provided for water quality control must be designed or selected to accommodate a specific flow rate corresponding to the first flush runoff volume.

A straightforward procedure for calculating first flush design flow rates is presented based on the MRM and intensity–duration–frequency equations that can be determined readily for locations in the United States covered by NOAA Atlas 14, Volumes 1 through 4. However, the same approach can be used wherever rainfall intensity–duration formulas like Equation (3) can be prepared. The solution uses dimensionless parameters, reducing the number of variables involved in the calculations. The resulting expression for the dimensionless storm duration that provides the desired flow rate requires an iterative solution. Graphs that provide sufficiently precise solutions for the intended purpose can be created to simplify the solution. However, application of the MRM is limited to catchments with drainage areas that are less than 12 ha (30 ac). For this reason, the procedure described in this investigation is restricted to small catchments.

Two examples are presented to show how first flush flow rates can be found where D_f = 6.35 mm (that is, where the first 6.35 mm or 0.25 in of runoff is to be treated), one in the Eastern United States (Raleigh, North Carolina) covered by NOAA Atlas 14, Volume 2 [52], and the other one in the arid Southwestern United States (Albuquerque, New Mexico) covered by NOAA Atlas 14, Volume 1 [51]. The method helps us to design first flush treatment facilities for small catchments, which are usually much less than 12 ha in size, where more complicated approaches are unnecessary. These examples illustrate the practical applicability of the method, which can be extended or adapted to different geographic locations, for designing first flush treatment facilities where more complicated approaches are unnecessary.

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