Modified rational method for sizing infiltration structures

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Abstract: A simple method is presented to size infiltration structures like infiltration basins and trenches to control storm water runoff. The runoff hydrograph is assumed to be trapezoidal in shape with a peak runoff rate calculated using the rational formula. Given the watershed time of concentration and the allowable runoff rate, the method determines the required size of the infiltration structure. A practical application section is included to demonstrate the use of the method.

Key words: rational method, infiltration basin, infiltration trench, capture volume, storage time.

Résumé : Une méthode simple est présentée afin de dimensionner des structures d'infiltration telles que des bassins ou des tranchées d'infiltration pour le contrôle des écoulements d'eau de pluies. L'hydrogramme d'écoulement est assumé être de forme trapèzoïdale avec un taux d'écoulement de pointe calculé en utilisant la formule rationnelle. Pour un temps de concentration du bassin et un taux d'écoulement permis donnés, la méthode détermine les dimensions requises pour la structure d'infiltration. Une section avec une application pratique est incluse dans l'article afin de démontrer l'utilisation de cette méthode.

Mots clés : méthode rationnelle, bassin d'infiltration, tranchée d'infiltration, volume d'emmagasinement, temps de rétention.

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Introduction

Infiltration structures, such as infiltration basins and trenches, are constructed to capture storm water runoff and let it percolate into the underlying soil. These structures are feasible where the soil has adequate permeability and the maximum water table (and (or) bedrock) elevation is sufficiently low. Unlike detention basins, there do not exist widely accepted design standards and procedures for infiltration practices. Generally, an infiltration structure is designed to store a "capture volume" of runoff for a period of "storage time".

Like all storm water control structures, the design of infiltration structures involves the calculation of a runoff hydrograph corresponding to a design storm. A number of computer-based rainfall runoff models, including the TR-20 (U.S. Department of Agriculture 1982) and HEC-1 (U.S. Army Corps of Engineers 1990), have been popular in the past. A much more versatile and user-friendly model, HEC-HMS (U.S. Army Corps of Engineers 2001), is now available for rainfall-runoff calculations. HEC-HMS includes a variety of options for design storm selection, separation of losses from rainfall, and runoff calculations.

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For watersheds not exceeding 8–12 ha in size, the rational method, which can be viewed as a simple rainfall-runoff model, has also found widespread use in drainage design. This method lumps all the watershed characteristics into a runoff coefficient and a time of concentration. In addition, the method assumes that the design storm intensity remains constant over the duration. As a result of these oversimplifications, the method should be used cautiously and only where a more complex model is not warranted in terms of the required accuracy, availability of data, and the project budget.

The rational method was originally meant for sizing hydraulic structures such as storm sewers and culverts that are designed to accommodate a peak discharge. The method was not meant for design problems involving runoff volumes. However, different variations of the rational method have been reported in the recent years for sizing detention basins successfully to serve small areas. One of these modified rational methods, originally presented by Aron and Kibler (1990), was also included in the *Hydrology Handbook* of the American Society of Civil Engineers (1996) and is expected to find widespread use. The method presented herein extends the Aron and Kibler (1990) procedure to sizing infiltration structures.

Capture volume

As suggested by Aron and Kibler (1990), the post-development runoff hydrograph is approximated by a trapezoid in the proposed method as shown in Fig. 1. The equilibrium is reached at $t = T_c$, where t is time and T_c is the time of concentration of the urban watershed. The equilibrium

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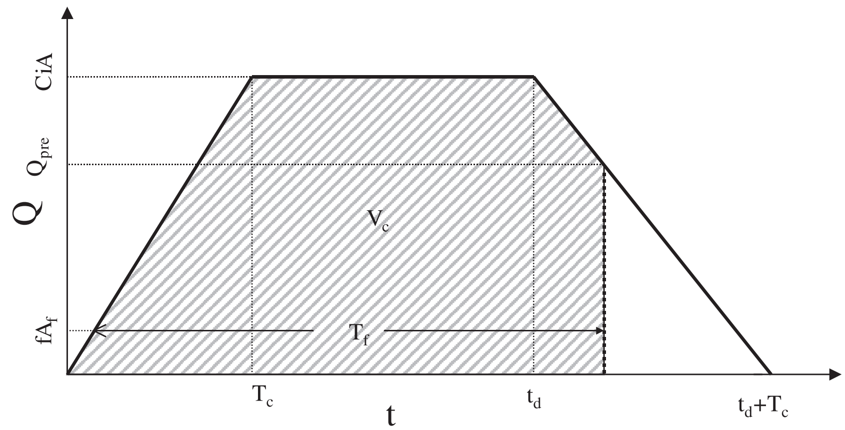
Fig. 1. Capture volume and filling time.



discharge is set equal to CiA, where C is the rational method runoff coefficient, i is the rate of design rainfall, and A is the watershed area. The state of equilibrium continues until $t = t_d$, where t_d is the design storm duration. After the rain stops at $t = t_d$, the runoff decreases linearly to zero at $t = t_d + T_c$.

Most storm water management programs require that the post-development peak discharge be reduced to the pre-development peak for the design return period by a best management practice. Therefore, an infiltration practice needs to capture that part of the runoff occurring before the runoff rate falls to a value equal to the pre-development discharge (Maryland Department of Natural Resources 1984). The shaded area shown in Fig. 1 represents the capture volume, V_c. From the geometry,

$$[1] \qquad V_c = CiAt_d - \frac{Q_{pre}^2 T_c}{2CiA}$$

where Q_{pre} is the pre-development peak discharge or the allowable peak discharge. The rate of rainfall, i, and the duration, t_d, are related through the intensity–duration–return period relationships for the project location. For a given design return period, an infinite number of combinations of i and t_d are possible. The i and t_d pair leading to the largest V_c is the most critical. As demonstrated in the practical application section below, a trial-and-error procedure is needed to identify the critical values of i and t_d.

Mathematical expressions are sometimes available to describe the intensity–duration–return period relationships. A commonly used form for a given return period is

$$[2] \qquad i = \frac{a}{b + t_d}$$

where a and b are fitting parameters. If such a relationship is available, we can eliminate the trial-and-error procedure to determine the critical storm duration and intensity. In that event, we substitute eq. [2] into eq. [1], differentiate the resulting equation with respect to t_d and set it equal to zero, and simplify to obtain the critical storm duration as

$$[3] \qquad t_d = \frac{CAa}{Q_{pre}} \sqrt{\frac{2b}{T_c}} - b$$

Filling time

Infiltration starts as soon as the runoff reaches the infiltration structure. In the current design practice, it is assumed that the infiltration takes place at a constant rate (Maryland Department of Natural Resources 1984; Federal Highway Administration 1996). The structure starts filling when the runoff rate reaching the structure exceeds fA_f, where f is the infiltration rate and A_f is the effective horizontal area over which infiltration occurs. The structure continues to fill until the runoff rate falls to a value equal to Q_{pre}, as shown in Fig. 1. The filling time, T_f, can be expressed as

$$[4] \qquad T_f = t_d + T_c - \frac{Q_{pre} T_c}{CiA}$$

Equation [4] should also include an additional term, $-(fA_f T_c)/(CiA)$, on the right-hand side, but this term is usually negligible.

Sizing an infiltration structure

Once the capture volume, V_c, and the filling time, T_f, are determined, an infiltration structure can be sized employing the procedures used in the current engineering practice. Details of these procedures are available in the literature (e.g., Maryland Department of Natural Resources 1984; Federal Highway Administration 1996). The general criteria are (*i*) the infiltration structure should drain completely within a specified storage time, (*ii*) the bottom of the infiltration structure should be at least a specified distance above the high water table level, and (*iii*) the infiltration structure should have enough storage capacity to accommodate the capture volume plus the rain falling directly over the structure minus infiltration during the filling time.

The criteria (*i*) and (*ii*) for infiltration structures to be excavated into the ground define an upper limit for the depth of the basin, respectively, as

$$[5] \qquad d_{max} = \frac{fT_s}{n}$$

and

$$[6] \qquad d_{max} = GW - h_{req}$$

Table 1. Practical application.

t_d (min) (1)	i (cm·h^{-1}) (2)	t_d (s) (3)	i (m·s^{-1}) (4)	$CiAt_d$ (m^3) (5)	$Q^2_{pre}T_c/(2CiA)$ (m^3) (6)	V_c (m^3) (7)
30	5.54	1800	1.5389×10^{-5}	1330	305	1025
40	4.74	2400	1.3167×10^{-5}	1517	356	1161
50	4.14	3000	1.1500×10^{-5}	1656	408	1248
60	3.67	3600	1.0194×10^{-5}	1762	460	1302
70	3.30	4200	9.1667×10^{-6}	1848	511	1337
80	3.00	4800	8.3333×10^{-6}	1920	563	1358
90	2.75	5400	7.6389×10^{-6}	1980	614	1366
100	2.53	6000	7.0278×10^{-6}	2024	667	1357
110	2.35	6600	6.5278×10^{-6}	2068	718	1350
120	2.20	7200	6.1111×10^{-6}	2112	767	1345

where $_{dmax}$ is the maximum allowable depth for the infiltration practice, f is the infiltration capacity of the underlying soil, which is assumed to remain constant, T_s is the storage time, n is the porosity of the aggregate material filling the infiltration structure such as infiltration trenches, GW is the high water table elevation, and h_{req} is the minimum required distance between the bottom of the infiltration practice and high water table. The smaller of the two d_{max} obtained from eqs. [5] and [6] will govern. For infiltration basins, $n = 1.0$.

For a trapezoidal structure having a rectangular bottom area of W times L and side slopes of z:1 (horizontal over vertical), criterion (*iii*) yields

$$[7] \qquad n[LWd + (L+W)zd^2 + \frac{4}{3}z^2d^3] = V_c + (L+2zd)(W+2zd)P - (L+zd)(W+zd)fT_f$$

where d is the depth of the infiltration structure and P is the rain falling directly over the structure. Equation [7] assumes that the effective infiltration area is equal to the horizontal area of the structure at mid-depth. Again, for infiltration basins, $n = 1.0$.

Practical application

An urban watershed has a drainage area of $A = 80\,000$ m^2, a runoff coefficient of $C = 0.60$, and a time of concentration of $T_c = 30$ min = 1800 s. An infiltration basin is being planned to reduce the peak discharge resulting from a 10-year design storm to $Q_{pre} = 0.5$ m^3·s^{-1}. The intensity–duration relationship for the 10-year events is tabulated in the first two columns of Table 1. The infiltration basin will be excavated into a sandy loam that has an infiltration capacity of $f = 2.5$ cm·h^{-1}. It is required that the bottom of the basin be at least $h_{req} = 1.2$ m above the water table. The water table elevation is GW = 4 m below the ground surface. The side slope of the basin will be 3 horizontal over 1 vertical, that is $z = 3$, and the infiltration basin is to drain completely within $T_s = 72$ h.

To determine the design capture volume, V_c values are calculated for the different t_d and i pairs tabulated in columns 1 and 2 of Table 1. In order to use consistent units, the t_d and i values are first converted to seconds and metres per second, respectively, and listed in columns 3 and 4. The corresponding values of V_c, tabulated in column 7, are calculated using eq. [1]. The largest V_c (1366 m^3) results from a storm of 90-min duration and 2.75 cm·h^{-1} intensity. Therefore, the critical storm duration is 90 min. The corresponding filling time is determined by using eq. [4] as being $T_f = 4745$ s = 79 min. Also, the depth of rain falling directly over the infiltration basin is $P = it_d = (7.639 \times 10^{-6}$ m·s^{-1})(5400 s) = 0.041 m.

We should note that we can fit an expression in the form of eq. [2] to the intensity–duration data given in columns 1 and 2 of Table 1 with $a = 327$ cm·min·h^{-1} = 327/[(100)(60)] = 0.0546 m and $b = 29$ min = 1740 s. Then by using eq. [3], we find $t_d = 5548$ s = 92.5 min. The corresponding rainfall intensity is $i = 2.69$ cm·h^{-1} = 7.476×10^{-6} m·s^{-1} and then the capture volume is $V_c = 1357$ m^3. This result is very close to the one obtained by trial-and-error. The small discrepancy is due to the use of discrete values of t_d and i.

To size the infiltration basin, we calculate the upper limit for the basin depth using eqs. [5] and [6], respectively, as d_{max} = (0.025 m·h^{-1})(72 h)/(1.0) = 1.8 m and d_{max} = 4.0 –1.2 = 2.8 m. Therefore, the depth of the infiltration basin should not exceed 1.8 m. Let us set $d = 1.8$ m. Then any L and W combination satisfying eq. [7] with $n = 1.0$, $d = 1.8$ m, $z = 3.0$, $V_c = 1366$ m^3, $P = 0.041$ m, $f = 2.5$ cm·h^{-1} = 6.944×10^{-6} m·s^{-1}, and $T_f = 79$ min = 4740 s is acceptable. For instance, if we pick $W = 20$ m, eq. [7] will yield $L = 24.55$ m. Let us choose $L = 25$ ft for practicality. Thus, the infiltration basin will be 1.8 m deep, and it will have side lengths of 20 and 25 m at the bottom. At the ground surface, the side lengths will be [20 + (2)(3)(1.8)] = 30.8 m and [25 + (2)(3)(1.8)] = 35.8 m.

Concluding remarks

Most infiltration structures are designed to serve areas that are smaller in size than the upper limit of 8–12 hectares established for the general applicability of the rational method. Therefore, the rational method appears to be suitable for infiltration structures. However, it should not be used where more complex procedures are warranted.

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