CAP-ORIFICE AS A FLOW REGULATOR FOR RAIN GARDEN DESIGN

James C.Y. Guo
Professor and Director, Civil Engineering, U of Colorado Denver.
Email: James.Guo@UCDenver.edu

Abstract
Rain gardens have been recognized as an effective device for on-site runoff volume reduction and storm water quality enhancement. A rain garden is designed as an infiltration basin with a shallow, wide water storage volume. The sub-base underneath the basin bottom is structured as a two-layered filtering medium. The upper sand layer provides the required filtering process and then the lower gravel layer provides a reservoir for a gradual release of the stored water through a perforated underdrain pipe. A newly constructed rain garden can have a high infiltration capacity. After several years of service, the accumulation of solids intercepted in the filtering layer will develop a clogging effect or the infiltration capacity in the sub-base continues decreasing. When the drain time exceeds the safety criteria, the rain garden needs to be replaced. Coping with a decaying infiltration rate, a rain garden needs to reduce its flow release in the early years. In this study, it is suggested that a cap orifice be installed at the exit of the underdrain pipe. This cap orifice is sized to reduce the initial infiltrating rate in the early years and then to work with the clogged infiltration rate throughout the life cycle of a rain garden. The design methodology presented in this paper allows the engineer to size this cap orifice according to the decay of infiltration rate, required drain time, and allowable flow release.

KEY WORDS: Rain Garden, Landscaping Detention, Low Impact, Infiltration, Stormwater

INTRODUCTION

On-site stormwater detention is an effective means for Low Impact Development (LID) designs. The design capacity for on-site storage basin is aimed at the micro events that are more frequent than the 3-month event (Guo and Urbonas 2002), and expected to be overtopped during the extreme event (Guo 2002). Aesthetically, a rain garden or also termed porous landscaping detention basin (PLDB), is designed to provide an infiltrating bed that is blended with landscaping bushes and vegetations. Hydrologically, a rain garden is operated as a surface-subsurface system for the purposes of on-site storm runoff quality and quantity control (Hunt wt. al. 2006).

As recommended shown in Figure 1, a rain garden provides a storm runoff detention capacity up to a depth of 6 to 12 inches in the infiltration basin (UDFCD Manual in 2010). The stored water should be evenly spread over a flat basin floor for infiltration. The sub-base media underneath the infiltration bed are structured as a two-layered filtering system that consists of an 18-inch (or 45.7 cm) sand-mix layer and another 8-inch (or 20.3 cm) gravel layer. The sand-mix material can be a combination of 85% of sand, 8% of shredded paper, and 7% compose (Guo et al. 2009). A perforated underdrain pipe runs through the gravel layer to collect and deliver the seepage flow to the downstream manhole. For a large infiltration bed, a set of parallel underdrain pipes needs to be laid with a spacing of 20 to 30 feet apart. The seepage flow between two underdrain pipes mainly moves downward into the perforated holes on the underdrain pipe.
The performance of a rain garden has to satisfy two basic goals, including storm water quality enhancement and the flow release control set forth in the local stormwater detention policy. As a tradeoff exists between these two purposes, the filtering media shall be designed with a high infiltration rate to ensure a good hydraulic efficiency while the underdrain system shall have a slow release to replicate the pre-development hydrologic condition for purpose of waterway preservation (Guo and Hughes 2001). On top of this tradeoff, the infiltration rate through the filtering media continues decreasing due to the clogging effect developed over time. As reported, the newly constructed rain garden can have a high water infiltrating rate of 5 to 10 inch/hr (Ames et al. in 2001). When the clogging effect is significantly developed, the infiltrating rate may be reduced to 1.0 inch/hr (UDFCD 2010). Throughout the life cycle of a rain garden, how to regulate the infiltrating rate is a challenge in both design and operation.

In this study, it is suggested that a cap orifice, as illustrated in Figure 1, be installed at the exit of the underdrain pipe. Usually such a cap orifice can be accessed for maintenance or adjustment at a manhole. During the early years, the fresh infiltration rate through the filtering media is high and efficient. The cap orifice shall be installed at the exit of the underdrain pipe as a regulator to reduce the flow release in order to satisfy the required residence time for solid settlement and to meet the allowable release rate for waterway preservation. In this study, the design procedure for rain garden is modified to balance the energy principle among the flows through the two layers of filtering system and the proposed cap orifice. This method can further be extended into the performance assessment through a rain garden’s life cycle when the clogging is developed through the filtering layers.

**HYDRAULIC PRINCIPLES FOR RAIN GARDEN**

As a dual flow system above and below the infiltration bed, the flow movement through a rain garden can be analyzed using the principle of continuity between the infiltrating water flow on the infiltrating bed and the seepage flow through the sub-base media (Guo 2003). As illustrated in Figure 1, under the assumption of steady state, the infiltrating and seepage flows are described by the Darcy’s law as (Fitts 2002):

\[ Q = fA_b = K_s I_s A_B = K_s I_s A_B \]  \hspace{1cm} (1)

\[ T_D = \frac{Y}{f} \]  \hspace{1cm} (2)
in which \( Q \) = flow release from rain garden \( [L^3/T] \), \( f \) = infiltration rate \( [L/T] \) on rain garden’s bottom area, \( A_b \) = rain garden’s bottom area \( [L^2] \), \( K_s \) = hydraulic conductivity coefficient in \( [L/T] \) for sand-mix layer, \( I_s \) = hydraulic gradient through sand layer, \( K_g \) = hydraulic conductivity coefficient in \( [L/T] \) for gravel layer, \( I_g \) = hydraulic gradient through gravel layer, \( T_D \) = drain time in \( [T] \) for rain garden, and \( Y \) = water storage depth in \( [L] \) such as 12 inches (30.5 cm). From the laboratory test (Guo et al. 2009), the hydraulic conductivity coefficients for sand-mix and gravel layers are summarized in Table 1.

<table>
<thead>
<tr>
<th>Rain Garden Filtering Material</th>
<th>Hydraulic Conductivity Under Fresh Condition inch/hr</th>
<th>Minimum Hydraulic Conductivity Under Clogged Condition Inch/hr</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand-mix</td>
<td>2.50</td>
<td>1.0</td>
</tr>
<tr>
<td>Gravel</td>
<td>25.0</td>
<td>Not Applicable</td>
</tr>
</tbody>
</table>

Note: 1 inch=25.4 mm

Table 1 Hydraulic Conductivity Coefficients for Filtering Media

Design of rain garden involves uncertainty in selection of design parameters. For instance, the infiltration rate through the sand-mix layer varies from 10 to 15 inch/hr for a newly constructed basin, 3 to 5 inch/hr for a matured basin, and 1.0 inch/hr or less for a clogged to plugged basin (Kocman 2009). Below 1.0 inch/hr for infiltration will result in such prolonged inundation that the infiltrating bed needs a replacement. For design, a moderate infiltration rate, 1.0 to 3.0 inch/hr, is selected to meet the design criteria for both water quality and quantity control. Consequently, how to replicate the pre-development flow release during the early years of a rain garden’s operation becomes a challenge. In this study, it is suggested that a cap orifice be employed to regulate the flow release at the exit of the underdrain pipe. The operation of a rain garden is controlled by the least flow capacity among the infiltrating flow on the infiltration bed, the seepage flow through the filtering layers, and the orifice flow at the exit.

The complicated flow system can be solved by an iterative procedure to balance the principles of continuity and energy through the filtering media. Referring to Figure 1, the flow is driven by the available head in the system as:

\[
H_t = Y + H_s + H_g \tag{3}
\]

in which \( H_t \) = total hydraulic head in \([L]\), \( H_s \) = thickness of sand-mix layer in \([L]\) such as 18 inches (45.7 cm), and \( H_g \) = thickness of gravel layer in \([L]\) such as 8 inches (20.3 cm). The energy losses for the seepage flow through the sand and gravel layers are calculated as:

\[
\Delta h_s = I_s H_s \tag{4}
\]

\[
\Delta h_g = I_g H_g \tag{5}
\]

As a result, the residual head applied to the underdrain pipe and cap-orifice flows is:

\[
\Delta H = H_t - \Delta h_s - \Delta h_g \tag{6}
\]

in which \( \Delta h_s \) = energy loss in sand layer, \( \Delta h_g \) = energy loss in gravel layer, and \( \Delta H \) = total loss in the filtering layers. In comparison, the gravel layer has a much higher seepage capacity than the sand layer. During the laboratory test, it was observed that the gravel layer is often in a partially saturated condition (Kocman 2009). The perforated underdrain pipe through the gravel layer may be in a saturated or unsaturated condition, depending on with or without a cap orifice that can produce a tail water effect to the flow system.
Case 1: Without a Cap Orifice

Without a cap orifice, the perforated underdrain pipe is directly connected to the downstream manhole. At the underdrain exit, the flow pressure has to drop to the atmospheric pressure. As a result, the infiltration rate in Eq (2) must satisfy the balance of energy as:

\[
\Delta H = H_i - \Delta h_s - \Delta h_g = 0 \quad \text{without a cap orifice} \quad (7)
\]

It implies that without a cap orifice, the available headwater in the system dictates the flow capacity. Or it is a condition of no control in flow release. As a result, the rain garden may be drained at a release rate higher than the pre-development condition. Its operation may have a drain time shorter than the required residence time for stormwater filtering and solid settlement.

Case 2: With a Cap Orifice

To regulate the flow release, a cap orifice can be installed at the exit of the perforated underdrain pipe. A cap orifice backs up the flow system to cause saturation in the gravel layer (Shani et. a. 1996). In doing so, the flow release from the rain garden is regulated with the cap orifice. To satisfy the principle of energy, the friction loss through the underdrain pipe is computed as:

\[
\Delta h_N = kL \frac{N^2 Q^2}{D^{16/3}} \quad (8)
\]

in which \(\Delta h_N\) = friction loss in [L] through underdrain pipe, \(L\) = pipe length in [L], \(D\) = diameter in [L] of underdrain pipe, \(N\) = Manning’s roughness coefficient, \(k\)=4.65 for unit of feet-second or10.28 for unit of meter-second. The cross section area for the required cap orifice is calculated as:

\[
A_o = \frac{Q}{C_d \sqrt{2g (H_i - \Delta h_s - \Delta h_g - \Delta h_N)}} \quad (10)
\]

in which \(A_o\) = opening area of cap orifice in \([L^2]\), \(C_d\) = discharge coefficient, and \(g\) = gravity acceleration in \([L/T^2]\). In practice, the cap orifice must have a diameter smaller than the underdrain pipe.

DESIGN EXAMPLE

A rain garden is designed to have an infiltration basin and two-layered filtering system. The infiltration bed for a rain garden has an flat area as \(A_B = 500 \text{ ft}^2\). Referring to Figure 2, the dimensions of filtering system are: \(Y=12 \text{ inches}, H_s=18 \text{ inches}, H_g=8 \text{ inches}\). The infiltration rate for the filtering media is estimated to decay from 10.0 to 1.0 inch/hr. The hydraulic conductivity is 2.5 inch/hr for the sand layer and 25.0 inch/hr for the gravel layer. Without an exit flow regulator, the flow rate released from this rain garden is determined using a trial-and-error procedure. Let us start with a guessed infiltrating rate at 5.0 inch/hr.

\[
Q = f A_B = \frac{5.0}{12 \times 3600} \times 500 = 0.058 \text{ cfs}
\]

The energy gradients through the two filtering layers are computed as:

\[
I_s = \frac{f}{K_s} = \frac{5.0}{2.5} = 2.0 \quad \text{for the sand layer}
\]
\[
I_g = \frac{f}{K_g} = \frac{5.0}{25.0} = 0.2 \text{ for the gravel layer}
\]

The energy losses through the sand and gravel layers are calculated as:

\[
\Delta h_s = I_s H_s = 2.0 \times 18 = 36.0 \text{ inches}
\]

\[
\Delta h_g = I_g H_g = 0.2 \times 8 = 1.6 \text{ inch}
\]

\[
H_f = Y + H_s + H_g = 12.0 + 18.0 + 8.0 = 38.0 \text{ inches}
\]

\[
\Delta H = H_f - \Delta h_s - \Delta h_g = 38.0 - 36.0 - 1.6 = 0.4 \text{ inch close to zero}
\]

With \( f=5.0 \text{ inch/hr} \), the total energy loss is 37.6 inches in comparison with the total available energy of 38 inches in the system. Therefore, the unregulated flow rate through the rain garden is determined to be 5.1 inch/hr after the second iteration.

Based on the pre-development release from the site, the flow release from this rain garden is no more than 3.0 inch/hr. The task is to design a cap orifice that will reduce the flow release from 5.1 to 3.0 inch/hr. Repeating the above procedure with \( f=3.0 \text{ inch/hr} \), the cap-orifice is determined as:

\[
Q = fA_p = \frac{3.0}{12 \times 3600} \times 500 = 0.035 \text{ cfs}
\]

The energy gradients through the two filtering layers are computed as:

\[
I_s = \frac{f}{K_s} = \frac{3.0}{2.5} = 1.2 \text{ for the sand layer}
\]

\[
I_g = \frac{f}{K_g} = \frac{3.0}{25.0} = 0.12 \text{ for the gravel layer}
\]

The energy losses through the sand and gravel layers are calculated as:

\[
\Delta h_s = I_s H_s = 1.2 \times 18 = 21.6 \text{ inches}
\]

\[
\Delta h_g = I_g H_g = 0.12 \times 8 = 0.96 \text{ inch}
\]

Considering the underdrain pipe is described as: \( D=4 \text{ inch} \), \( L=25 \text{ feet} \), and \( N=0.012 \), the friction loss through the underdrain pipe is:

\[
\Delta h_N = 4.62L \frac{N^2Q^2}{D^{(16/3)}} = 4.62 \times 25 \times \frac{0.012^2 \times 0.035^2}{(4/12)^{(16/3)}} = 0.007 \text{ ft} = 0.084 \text{ inch}
\]

With \( C_d=0.70 \), the cross sectional area for the cap orifice is calculated as:
\[
A_o = \frac{0.035}{0.70 \sqrt{2 \times 32.2(38 - 21.6 - 0.96 - 0.084)/12}} = 0.0055 \text{ sq ft or one inch in-diameter.}
\]

Repeating the same process, Table 2 presents a comparison among various sizes of cap orifice for different allowable release rates. As expected, the smaller flow release from the rain garden leads to a smaller cap orifice and a longer drain time.

<table>
<thead>
<tr>
<th>Case</th>
<th>Infiltration rate</th>
<th>Drain time</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>inch/hr</td>
<td>in hours</td>
</tr>
<tr>
<td>No cap orifice</td>
<td>5.1</td>
<td>2.4</td>
</tr>
<tr>
<td>1.0-inch cap orifice</td>
<td>3.0</td>
<td>4.0</td>
</tr>
<tr>
<td>0.75-inch cap orifice</td>
<td>2.0</td>
<td>6.0</td>
</tr>
</tbody>
</table>

**Table 2: Drain Time Controlled by Cap Orifice**

As time goes on, the clogging effect will be developed in the sand-mix layer. As listed in Table 3, the reduced hydraulic conductivity in the sand layer from 2.5 to 0.5 inch/hr represents the clogging stages (Li and Davis 2008). With a decayed infiltration rate, the drain time becomes prolonged. Using the aforementioned procedure, the change of drain time can be simulated using the decreased infiltrating rate. With a one-inch cap orifice, Table 3 presents the predicted drain times at the various stages through the life cycle of the rain garden. As recommended, the rain garden needs to be replaced after the infiltration rate become less than 1.0 inch/hr (UDFCD 2010). Such a low infiltration rate results in inundation in the infiltration bed longer than 12 hours. Prolonged standing water is also considered as a hazard to the public.

<table>
<thead>
<tr>
<th>Rain Garden Condition</th>
<th>Sand Conductivity inch/hr</th>
<th>With one-inch Cap Orifice Reduced Infiltration Rate in/hr</th>
<th>Flow Release cfs</th>
<th>Drain Time hr</th>
</tr>
</thead>
<tbody>
<tr>
<td>New</td>
<td>2.500</td>
<td>3.00</td>
<td>0.033</td>
<td>4.0</td>
</tr>
<tr>
<td>Decayed</td>
<td>1.500</td>
<td>2.28</td>
<td>0.026</td>
<td>5.3</td>
</tr>
<tr>
<td>Clogged</td>
<td>1.000</td>
<td>1.43</td>
<td>0.017</td>
<td>8.4</td>
</tr>
<tr>
<td>Plugged</td>
<td>0.500</td>
<td>1.00</td>
<td>0.012</td>
<td>12.0</td>
</tr>
</tbody>
</table>

**Table 3: Drain Times through Life Cycle of 2-Layered Rain Garden**

**DESIGN SCHEMATICs**

The design example reveals that \(\Delta h_g\) and \(\Delta h_n\) are numerically negligible in comparison with \(\Delta h\), because the conductivity of sand-mix is much smaller than that of gravel. As a result, Eq (10) can be reduced to and normalized as:

\[
\frac{A_o}{A_H} = \frac{F_f}{C_d \sqrt{2(1 - \frac{\Delta h}{H_t})}} \tag{11}
\]

\[
F_f = \frac{f}{\sqrt{gH_t}} \tag{12}
\]
\[
\frac{\Delta h_s}{H_t} = \frac{f}{K_s} \frac{H_s}{H_t}
\]

(13)

where \(F_t\) = infiltration Froude Number. Eq 12 indicates that this system is characterized with the infiltration flow Froude number. Considering that \(C_d = 0.7\), Eq (11) is converted into a design chart in Figure 2.

For instance, the design example has an infiltration Froude number as:

\[
F_f = \frac{f}{\sqrt{gH_t}} = \frac{3.0/(12 \times 3600)}{\sqrt{32.2 \times 38/12}} = 6.88E - 06
\]

\[
\frac{\Delta h_s}{H_t} = \frac{f}{K_s} \frac{H_s}{H_t} = \frac{3.0}{2.5} \frac{18}{38} = 0.57
\]

From Figure 2, the area ratio is found to be 10.5E-06 or \(A_o = 0.0055\) sq ft for the cap orifice.

**CONCLUSIONS**

In this study, a one-dimension approach was developed to design a two-layered rain garden with a cap orifice at the underdrain exit to regulate the infiltration rate. The procedure satisfies both the continuity of flows and the balance of energy through the filtering layers underneath the rain garden. This method is applicable to the infiltrating flows between two parallel perforated sub-drain pipes, 20 to 30 feet apart,
because the vertical flow component dominates the flow movement. In case of a complicated sub-drain system, a two-dimension method shall be considered.

The design chart produced using Eq 11 does not work with a one-layered filtering system because $\Delta h_s = H$ results in indefinite. It is critically important that a gravel layer exists below the sand layer because it provides storage and suction effects when a cap orifice backs up the flow. Mathematically, Eq 11 verifies the effective operation of two-layers filtering system for rain garden design.

The recommended or standardized dimensions for rain garden design directly dictate its flow release through the pre-selected filtering system. It is essential that a cap-orifice be installed and sized to closely mimic the pre-development release and also satisfy the required drain time to avoid prolonged inundation in the basin. Although a cap-orifice is sized to reduce the infiltrating rate in the early years of service, removing such a cap orifice after the sub-base system is significantly clogged would not increase the infiltrating capacity.

REFERENCES


Appendix II

$A_B$ = RAIN GARDEN’s bottom area [L$^2$],
$A_o$ = opening area of cap orifice in [L$^2$],
$C_d$ = discharge coefficient
$D$ = diameter in [L] of underdrain pipe,
$f$ = infiltration rate [L/T] on RAIN GARDEN’s bottom area,
$F_f$ = infiltration Froude Number
$g$ = gravity acceleration in [L/T$^2$].
$H_s$ = thickness of sand-mix layer in [L] such as 18 inches (45.7 cm),
$H_g$ = thickness of gravel layer in[L] such as 8 inches (20.3 cm),
$H_t$ = total hydraulic head in [L].
$\Delta h_s$ = energy loss in sand layer,
$\Delta h_g$ = energy loss in gravel layer,
$\Delta H$ = total loss in the filtering layers
$\Delta h_N$ = friction loss in [L] through underdrain pipe,
$I_s$ = hydraulic gradient through sand layer,
$I_g$ = hydraulic gradient through gravel layer,
k = 4.65 for unit of feet-second or10.28 for unit of meter-second.
$K_s$ = hydraulic conductivity coefficient in [L/T] for sand-mix layer,
$K_g$ = hydraulic conductivity coefficient in [L/T] for gravel layer,
$L$ = pipe length in [L],
$N$ = Manning’s roughness coefficient.
$Q$ = flow release from RAIN GARDEN [L$^3$/T],
$T_D$ = drain time in [T] for RAIN GARDEN,
$Y$ = water storage depth in [L] such as 12 inches (30.5 cm)