Numerical Modeling Calibration and Prediction for Rain Garden Operation

by
Toan Luu
And
James Guo

UNIVERSITY OF COLORADO DENVER
College of Engineering and Applied Sciences
Hydrology and Hydraulics Graduate Program
Department of Civil Engineering
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Calibration and Prediction of Flows through Rain Garden

1. Objective:

To calibrate the hydrologic parameters for the as-built Rain Garden located at 21st and Iris, Lakewood, CO, and to determine the necessary adjustment on its operation.

2. Procedure

- Review and select the rainfall-runoff measurements in field
- Develop a subsurface hydrologic model to simulate the seepage flows through the filtering layers underneath the rain garden.
- Identify and calibrate the system parameters
- Make a flow adjustment using a cap-orifice.
- Predict the flow release from the rain garden.

Field Measurements at the Test Site

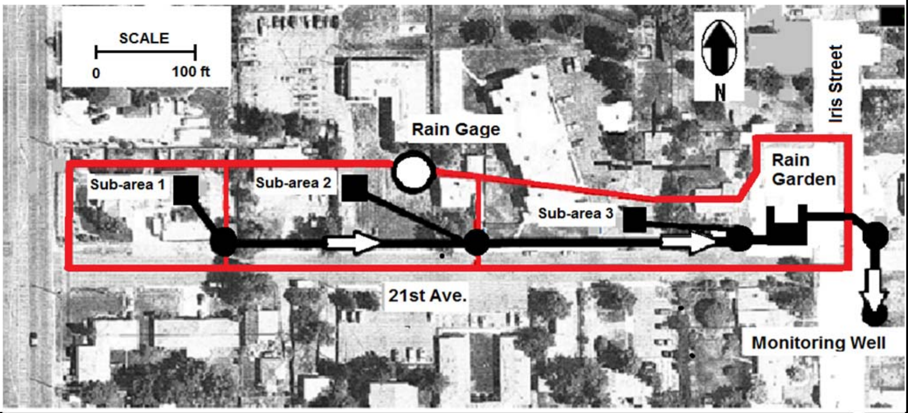
- Type of LID: rain garden
- Location: 21st Ave. & Iris St., Lakewood, Colorado.
- Tributary area: 1.9 acres , impervious area : 47%
- No street gutter
- Rainfall events:

1) May 18, 2011:
1.43 in. &13 hrs and 50 min.
2) July 7, 2011:
1.73 in. & 7 hrs and 5 min.
3) July 7, 2012
1.35 in. & 7 hrs
4) July 9, 2012
0.48 in. & 35 min.

Data source: Urban Drainage Flood Control District (UDFCD)

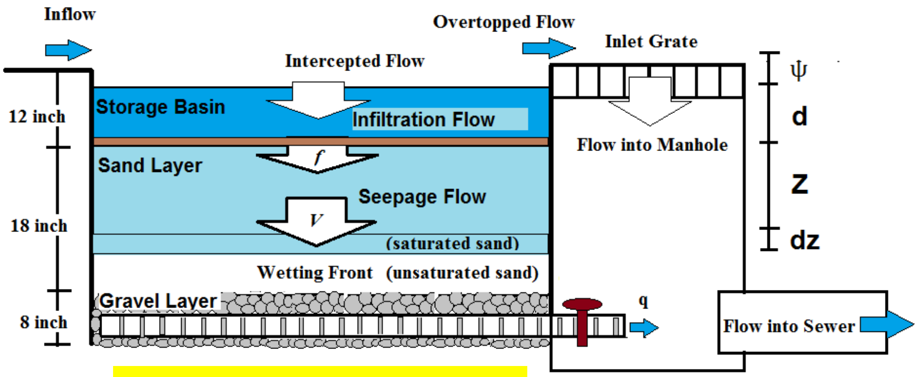
SWMM for on-surface Runoff Flow

- SWMM model
- The watershed boundary: red lines
- 3 sub-catchments
- Flow direction: west to east



Field Monitoring

Surface-subsurface Flow System



Flow System:
Flow Interception
Infiltrating Water on Surface
Seepage flow in sub-surface
Sub-drain flow through perforated pipe

Calibration Parameters:
Initial Soil Water Content
Hydro Conductivity

Surface and Subsurface Flows

- Infiltrating flow on basin bottom

$$f = \frac{dF}{dt} = \frac{\Delta\theta . dz}{dt} \qquad (1)$$

where F = cumulative infiltration, $\Delta\theta$ =moisture deficit

- Seepage flow through the saturated zone using the Darcy's law:

$$V = f = i.K_s = K_s\left(1 + \frac{\Delta h}{Z}\right) \qquad (2)$$

in which: $\Delta h = d - \psi$, d = ponding depth, ψ = suction head, K_s= hydraulic conductivity, V= Darcy velocity, i = hydraulic gradient

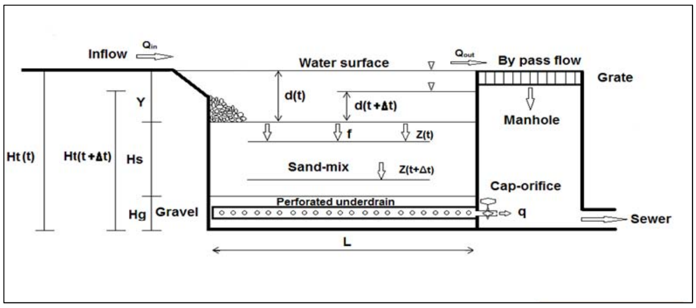
- Flow Continuity➔ Eq. (1)= Eq. (2)

$$\frac{K_S . dt}{\Delta\theta} = \frac{z.dz}{z+\Delta h} = \left(1 - \frac{\Delta h}{z+\Delta h}\right) dz \qquad (3)$$

Eq (3) describe the movement of the wetting front that starts from the basin bottom and marches downward till it reaches the gravel layer.

For each time step, Eq (3) shown the saturation process through the sand layer.

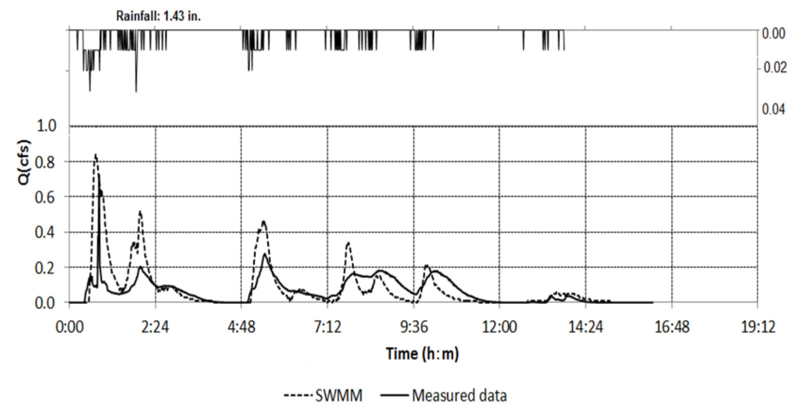
Numerical Steps for Flow Calibration



- $\Delta Z(t) = Z(t+\Delta t) - Z(t) = \frac{K_S . \Delta t}{\Delta\theta} + \Delta h(t) Ln\left(\frac{Z(t)+\Delta Z(t)+\Delta h(t)}{Z(t)+\Delta h(t)}\right)$ (5)
- $q_o(t+\Delta t) = K_s \frac{d(t+\Delta t)+H_S}{H_S} A_R$ (6)
- $d(t+\Delta t) = d(t) - \frac{q_o(t)\Delta t}{A_R} + \frac{(Q_{in}(t+\Delta t) - Q_{out}(t+\Delta t))\Delta t}{A_R}$ (7)
- $q_a(t+\Delta t) = C_d A_o \sqrt{2g\left(H_t(t+\Delta t) - \Delta h_s - \Delta h_g - \Delta h_N\right)}$ (8)
- $q(t+\Delta t) = min[q_o(t+\Delta t), q_a(t+\Delta t)]$ (9)

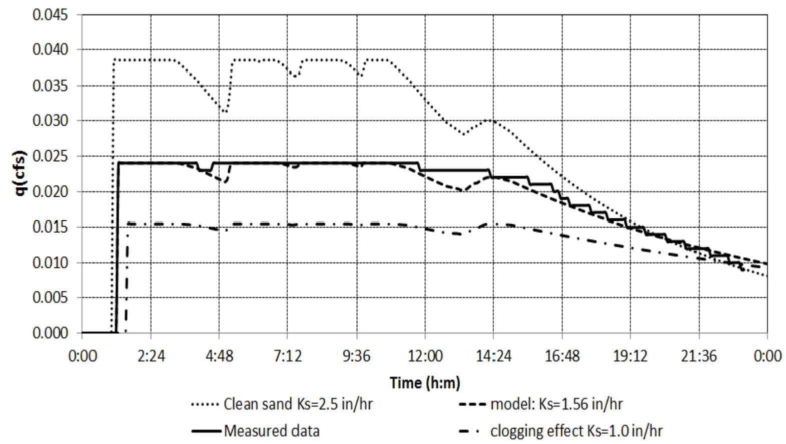
Loading of Runoff Flows from Watershed

May 18, 2011 Event



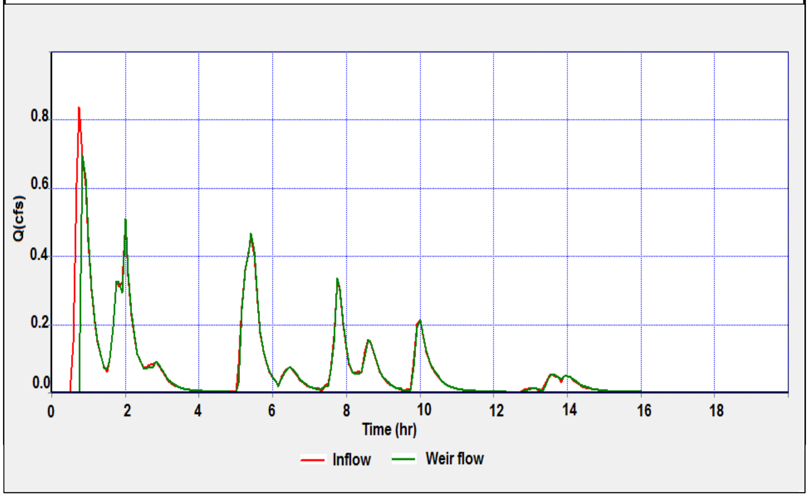
Calibration Parameters for Seepage Flows Through the Rain Garden

- 1) May 18, 2011 Event:



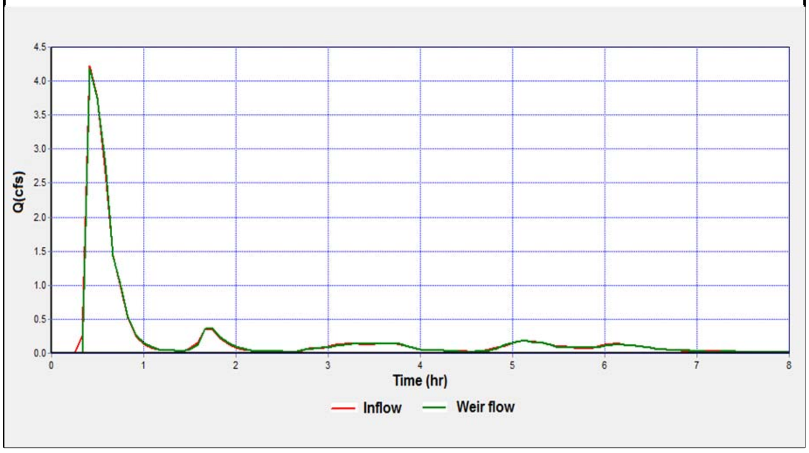
Intercepted Flow by Rain Garden
By-pass Flow overtopping the Weir

1) May 18, 2011 Event: 50 min.



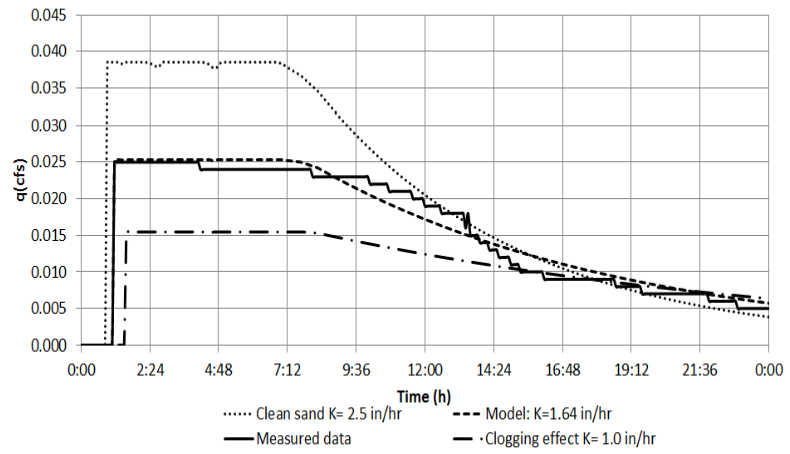
Intercepted Flow by Rain Garden
By-pass Flow overtopping the Weir

- 2) July 7, 2011 Event: 25 min.



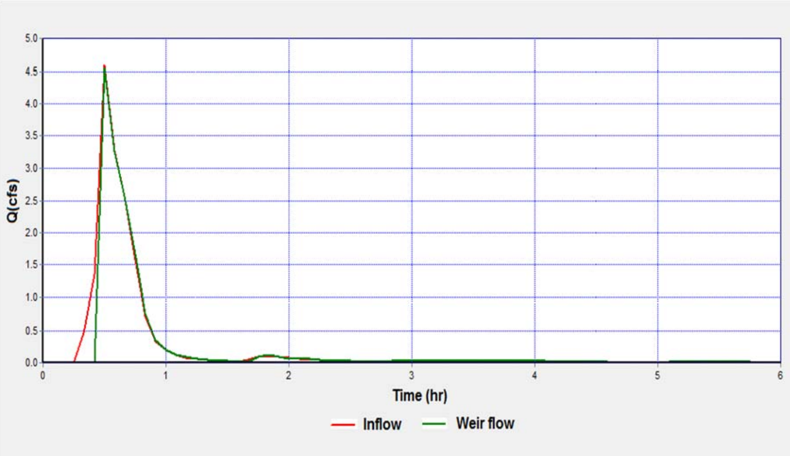
Seepage Outflow through Rain Garden

- 1) July 7, 2011 Event:



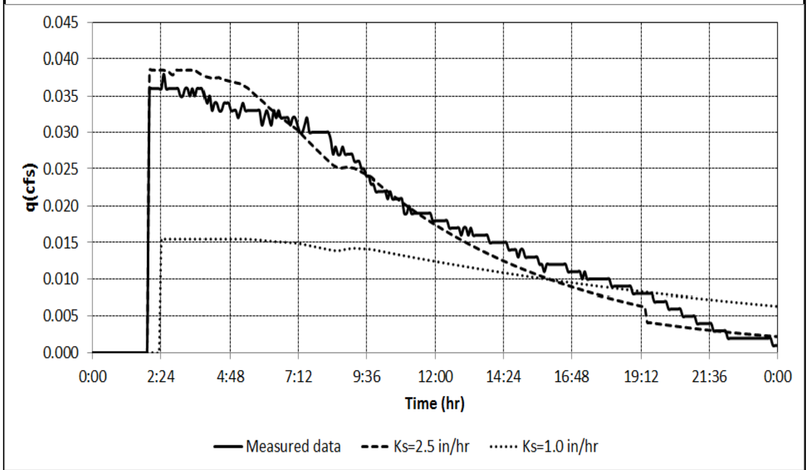
Intercepted Flow by Rain Garden
By-pass Flow overtopping the Weir

3) July 7, 2012 Event: 26 min



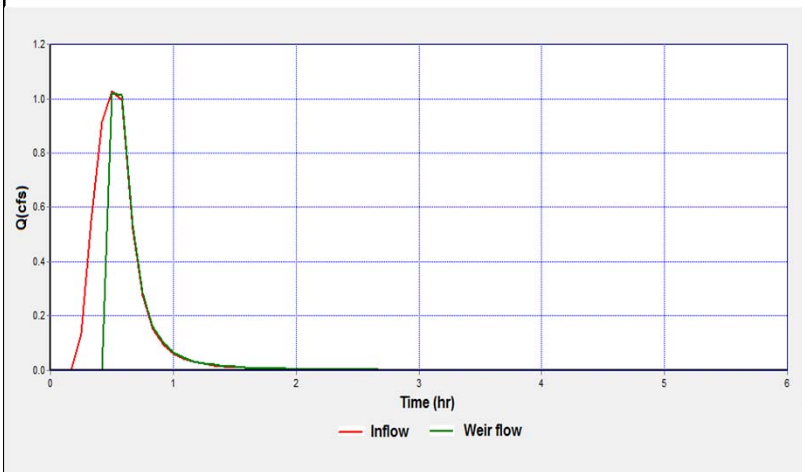
Seepage Outflow through Rain Garden

- 1) July 7, 2012 Event:



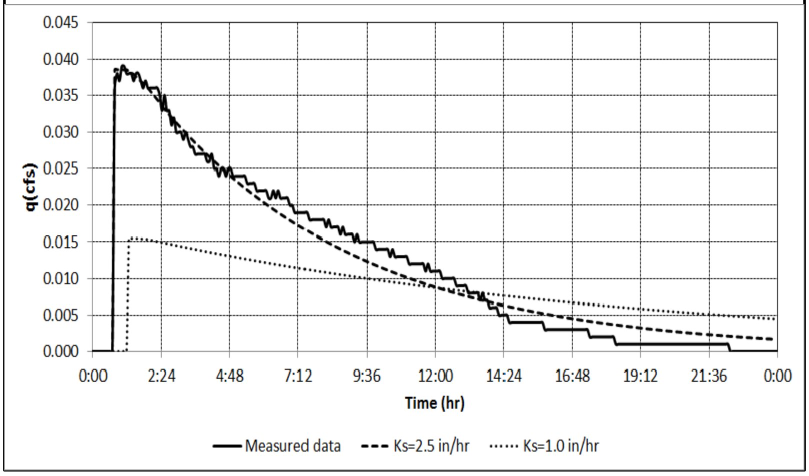
Intercepted Flow by Rain Garden By-pass Flow overtopping the Weir

4) July 9, 2012 Event: 26 min.



Seepage Outflow through Rain Garden

- 1) July 9, 2012 Event:



Calibration using MSE

- **MSE:** Mean Square Error

$$MSE = \frac{1}{n}\sum_{i=1}^{n}(\hat{Y} - Y_i)^2$$

- **Drain times derived from 4 events. None meets the design drain time of 12 hours.**

Event	MSE	Drain time
May 18, 2011	8.47E-07	5 hr 20 min.
July 07, 2011	2.01E-06	5 hr
July 07, 2012	3.99E-06	3 hr 15 min.
July 09, 2012	4.70E-06	3 hr 15 min.

- **Solution:** reduce the diameter of the cap orifice to limit the flow release.

Energy Principle for Seepage Flow – No cap

- $T_d = \frac{Y}{f} = \frac{12\ in.}{1.0\ in/hr} = 12\ hrs$ (10)
- Assume the flow is steady:

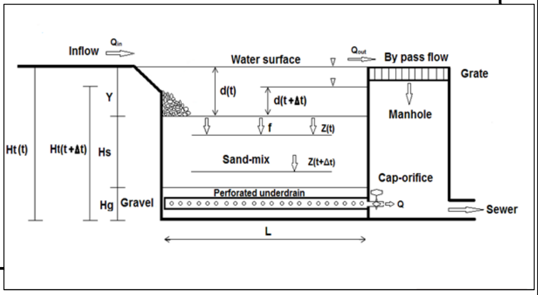
  $Q = fA_R = K_s I_s A_R = K_g I_g A_R$ (11)
- total hydraulic head

  $H_t = Y + H_s + H_g$ (12)
- Residual head:

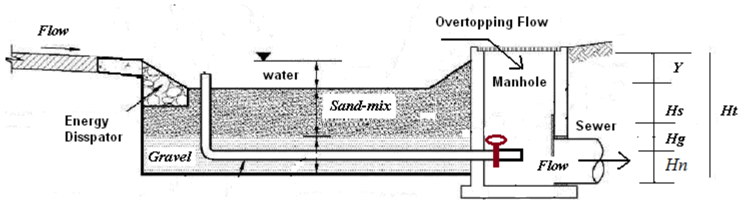
$$\Delta H = H_t - \Delta h_s - \Delta h_g - \Delta h_N$$

$$\Delta h_s = \frac{f}{K_s} H_s$$

$$\Delta h_g = \frac{f}{K_g} H_g$$



Flow adjustment using Cap-orifice



To satisfy the principle of energy, the friction loss through the underdrain pipe is computed as:

$$\Delta h_N = kL\frac{N^2Q^2}{D^{(16/3)}} \qquad (8)$$

in which Δh_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 or 10.28 for unit of meter-second. The cross section area for the required cap orifice is calculated as:

$$A_o = \frac{Q}{C_d\sqrt{2g(H_t - \Delta h_s - \Delta h_g - \Delta h_N)}} \qquad (10)$$

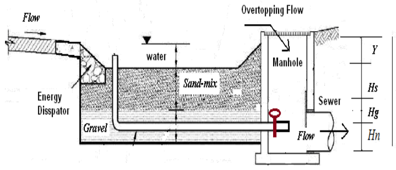
in which A_o = opening area of cap orifice in [L^2], C_d = discharge coefficient, and g= gravity acceleration in [L/T^2]. In practice, the cap orifice must have a diameter smaller than the underdrain pipe.

A rain garden is designed to release the ponding water depth of 12 inch over 4 hours using a flow regulator. The infiltration bed has an flat area of 500 ft^2. The dimensions of filtering system are: *Y=12 inches,* H_s*=18 inches,* H_g*=8 inches*. The hydraulic conductivity is 2.5 inch/hr for the sand layer and 25.0 inch/hr for the gravel layer. A cap-orifice is used as the flow regulator. Determine the opening area for the cap-orifice.

$$Q = fA_B = \frac{3.0}{12 \times 3600} \times 500 = 0.035 \ \text{cfs}$$

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

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



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 inch, L=25 feet, and N=0.012, the friction loss through the underdrain pipe is:

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

With C_d=0.70, the cross sectional area for the cap orifice is calculated as:

$$A_o = \frac{0.035}{0.70\sqrt{2 \times 32.2(38 - 21.6 - 0.96 - 0.084)/12}} = 0.0055 \ \text{sq ft} \ \text{or one inch in-diameter.}$$

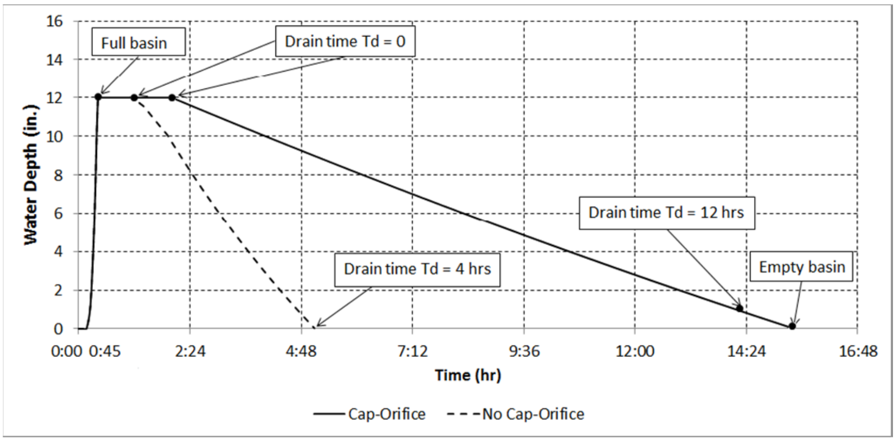
Operation of Cap-orifice for life cycle of rain garden

Case	Infiltration rate inch/hr	Drain time in hours
No cap orifice	5.1	2.4
1.0-inch cap orifice	3.0	4.0
0.75-inch cap orifice	2.0	6.0

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

Guo, James C.Y. (2012). "*Cap-orifice as a Flow Regulator for Rain Garden Design* ASCE J of Irrigation and Drainage Engineering, Vol. 138, No. 2, February.

Predictions of Seepage Flow with a Cap-orifice



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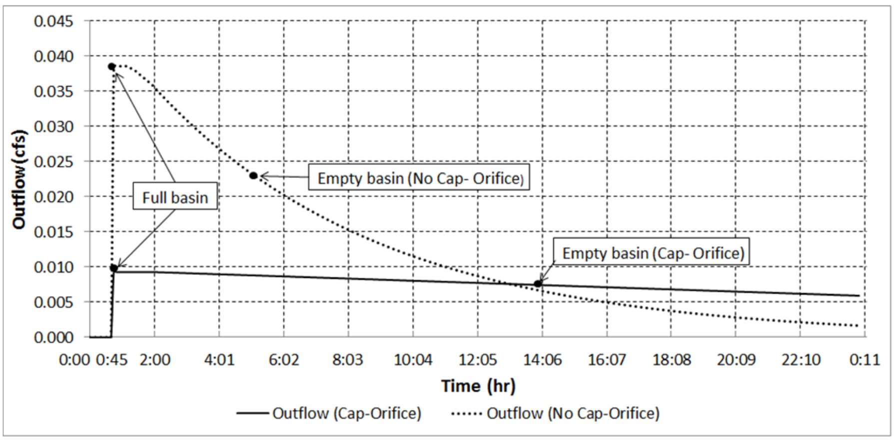

Q and A

FOR MORE INFORMATION

James.Guo@UCDenver.edu
WWW.UCDENVER.EDU/~Jguo -- Website
WWW.UDFCD.ORG -- Free Software
WWW.URBANWATERSHEDS.ORG – Training Classes

My kind of LID Island and cascading flows