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## SUSTAINABLE SURFACE-SUBSURFACE STORM WATER DETENTION SYSTEM

by

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**Abstract:** The public safety has directed the traditional approach to remove stormwater from streets as quickly as possible. Under this concept, an urban area is equipped with gutters, swales, and storm sewers. Many urban environmental studies have revealed that the traditional approach in storm water management has increased flooding and stream erosion. Increasing concerns have been arisen regarding how the traditional approach affects the balance of a water body, creates shock loads of pollutants to the receiving waters, and contributes to treatment plant malfunction. In response to various concerns, new concepts have been developed to cope with urban storm water. The concept of Stormwater Best Management Practices (BMP's) has begun to alter the designs of urban storm water systems. For instance, on-site permeable pavement, vegetal beds, dry wells rock-filled trench, open ditch, infiltrating basin, percolation basin, wetland, retention, and detention have been adopted as effective measures. Over the years, we have experienced some failures that have caused standing waters, prolonged draining process, and frequent clogging etc. Many of these problems were induced by the fact that the current design approach of storm water storage systems only focuses on surface hydrology and ignores the subsurface constraints. As a result, when the subsurface seepage rate cannot sustain the surface infiltrating rate, the system is backed up and fails to function well. This paper presents a new concept to combine subsurface and surface requirements into storm water basin designs.

**Keywords:** stormwater, flood, detention, retention, BMP's, infiltration seepage, wetland, trench, basin



Photo 1 Storm Water Detention Basins

### INTRODUCTION

A storm water storage system must be designed to control the release. The required detention volume for the basin under design is defined by the difference of the developed inflow volume and allowable release volume. On-site stormwater disposal relies on soil infiltration which is often determined by the surface soil texture. During the loading

period, the basin infiltrates water into the soils. The soil pores provide the soil-water storage volume. After the saturation, the infiltrating water must be sustained by the seepage rate that recharges the local groundwater table; otherwise, the water mounting will back up the soil-water system. Therefore, it is recommended that the storm water disposal basins have a storage volume no more than the on-site soil porosity, and its long-term operation must be sustained by the subsurface hydraulic transmissibility condition. Therefore, design of an on-site infiltration basin includes: (1) *determination of storage volume by surface hydrologic requirements*, (2) *selection of basin geometry by the storage capacity in the soil mediums*, (3) *evaluation of subsurface sustainability for a long-term (saturated) operation*.

## BASIN STORAGE VOLUME

At the basin site, the soil infiltration on the land surface and the design rainfall event dictate the storage volume for the basin. Secondly, the soil water storage capacity beneath the basin sets up the limit for the maximum water depth in the basin. Therefore, the design procedure begins with the storage volume and then the selection of basin geometry. Infiltration basin is often placed next to a small, highly paved catchment such as parking lots and business strip. Therefore, the Rational method is applicable to the peak flow prediction as:

$$Q_d = CI_d A \quad (1)$$

$$I_d = \frac{a}{(T_d + b)^n} \quad (2)$$

in which C = runoff coefficient, A= watershed area,  $I_d$  = rainfall intensity,  $T_d$  = rainfall duration,  $Q_d$ = peak runoff rate, and a, b, and n= constants on the Intensity- Duration- Frequency (IDF) formula. Soil infiltration can be described by the Horton's formula that states:

$$f(t) = f_c + (f_0 - f_c)e^{-kt} \quad (3)$$

in which  $f(t)$  = infiltration in inch/hr (or mm/hr) at time t after it rains,  $f_0$  = initial infiltration rate,  $f_c$  = final infiltration rate, and  $k$  = decay coefficient in 1/hr. Integration of Eq 3 yields:

$$F(t) = f_c t + \frac{(f_0 - f_c)}{k} (e^{-kt} - 1) \quad (4)$$

in which  $F(t)$ = infiltration depth in inch (or mm) at time t. The volume-based method is to find the maximum volume difference between the inflow and outflow volumes under a series of storm events with different durations. For a specified rainfall duration, the storm water storage volume is equal to:

$$V_d = \alpha CI_d AT_d - \beta A_b F(T_d) \quad (5)$$

in which  $V_d$  = storage volume,  $A_b$  = infiltrating area,  $\alpha$  and  $\beta$ = unit conversion factors. The maximal value of Eq 5 is achieved by setting its first derivative with respect to  $T_d$  equal to zero, and it results:

$$\frac{dV_d}{dT_d} = \left\{ CA\alpha \left[ \frac{-nT_d}{(T_d + b)^{n+1}} + \frac{1}{(T_d + b)^n} \right] - \beta A_b f(T_d) \right\} = 0 \quad \text{when } T_d = T_m \quad (6)$$

in which  $T_m$  = the design rainfall duration described by Eq 6. Solution of Eq 6 is:

$$T_m = \frac{1}{n} \left[ (T_m + b) - (T_m + b)^{n+1} \frac{\beta A_b}{a\alpha CA} f(T_m) \right] \quad (7)$$

When the value of  $b$  in Eq 7 is numerically negligible, the approximate solution of Eq 8 can be:

$$T_m = \left[ \frac{2\alpha a C A (1-n)}{\beta A_b f(T_m)} \right]^{\frac{1}{n}} \quad (8)$$

In fact, Eq 8 can also provide the first approximation to the solution of Eq 7 during the trial and error procedure. The maximum storage volume,  $V_m$ , is:

$$V_m = \alpha C I_m A T_m - \beta A_b F(T_m) \quad \text{at} \quad T_d = T_m \quad (9)$$

The average infiltration rate,  $f$ , throughout the storm duration is:

$$f = \frac{F(T_m)}{T_m} \quad (10)$$

**Example 1** In the City of Denver, Colorado, the 10-year storm intensity is described by Eq 2 with  $a=45.92$ ,  $b=10.0$  and  $n=0.786$ . A residential subdivision of 2.1 acre is to be developed with a runoff coefficient of 0.65. The 10-year storm runoff from this watershed will drain into 180-ft by 20-ft trench basin. The infiltration rates of the basin are:  $f_o= 6.50$  inch/hr,  $f_c= 1.80$  inch/hr, and  $k = 6.50$  /hour. Based on the given information, the unit conversion factors and the bottom area of the basin are:

$$\alpha = 60, \beta = 1/12, \eta = \frac{1}{12 \times 3600}, \text{ and } A_b = \frac{100.0 \times 36.0}{43560.0} = 0.083 \text{ acre}$$

Substituting these variables into Eq's 3 and 7 yields

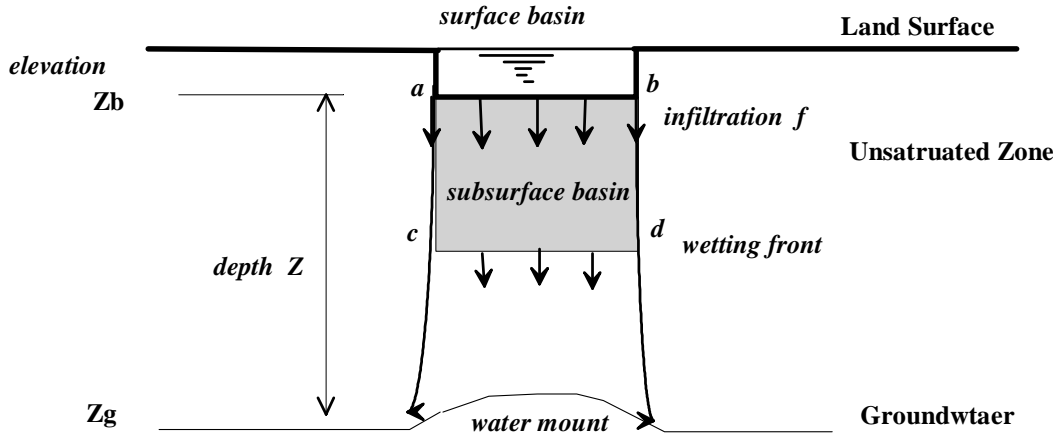
$$f(T_m) = 1.80 + (6.50 - 1.80)e^{-\frac{T_m}{60 \times 6.5}}$$

$$T_m = \frac{1}{0.786} \left[ (T_m + 10) - (T_m + 10)^{0.786+1} \frac{\frac{1}{12} \times 0.083 \times f(T_m)}{45.92 \times 60.0 \times 0.65 \times 2.1} \right]$$

The design storm duration described by these two equations was found to be:  $T_m=340.0$  minute. Using Eq 9, the required detention volume for this case is 0.219 acre-ft. The total infiltration depth,  $F(T_m)$ , calculated by Eq 4 is 4.33 inches.

## BASIN GEOMETRY

The above procedure yields a storage volume based on the surface hydrology without taking the subsurface condition into consideration. If the soil infiltration rate at the land surface is higher than the underground seepage rate, the system is backed up and may even cause a failure in the operation. To be conservative, the water storage volume in soil pores can serve as a limit for the water depth in the basin.



**Figure 1 Illustration of Soil Saturation beneath a Basin**

According to the diffusion theory, the seepage flow through the soil medium in Figure 1 can be described as (Green and Ampt, 1911):

$$\frac{\partial \theta}{\partial t} + \frac{\partial f}{\partial z} = 0 \quad (11)$$

in which  $\theta$  = soil moisture content,  $t$  = elapsed time,  $f$  = infiltration rate, and  $z$  = vertical distance below the basin. Consider the soil medium between the basin bottom and groundwater table as a control volume. The finite difference form of Eq 11 is:

$$\Delta \theta = \frac{\Delta f \Delta t}{\Delta z} \quad (12)$$

As illustrated in Figure 1, the value of  $\Delta \theta$  is the difference between the soil initial and saturated moisture contents. The value of  $\Delta z$  is the depth of the soil medium beneath the basin. The value of  $\Delta f$  is equal to the infiltration rate from the basin because there is no recharge to the groundwater table before the wetting front reaches the groundwater table. As a result, Eq 12 becomes:

$$(\theta_s - \theta_0) = \frac{(f - 0)(T_d - 0)}{(Z_b - Z_g)} = \frac{T_d f}{Z} \quad (13)$$

in which  $\theta_s$  = soil porosity,  $\theta_0$  = soil initial water content,  $Z_b$  = elevation at basin bottom,  $Z_g$  = elevation of groundwater table,  $T_d$  = drain time, and  $Z$  = distance to groundwater table. Re-arranging Eq 13, the drain time at the basin site is derived as:

$$T_d = \frac{Z(\theta_s - \theta_0)}{f} \quad (14)$$

Eq 14 indicates that the drain time of an infiltration basin is dictated by the storage capacity in the soil pores and the infiltration rate. And the water storage volume in the soil pores is equivalent to:

$$d = Z(\theta_s - \theta_0) \quad (15)$$

in which  $d$  = equivalent water depth in soil pores. Eq 15 sets the limit for the water depth in the basin. As a result, the basin area is:

$$A_o = \frac{V_m}{Z(\theta_s - \theta_o)} \quad (16)$$

Eq 9 defines the required storage volume in the basin; Eq 15 sets the maximum water depth in the basin, Eq 16 defines the minimum basin bottom area; and Eq 14 calculates the drain time to release the stored volume. The above design procedure applies to the soil mediums under an unsaturated condition. During an event, the storm water quality control basin may saturate the soil mediums. It is important to understand that the soil medium beneath a retention basin with a permanent pool or a long-term groundwater recharging pond would have saturated already. Under a saturated condition, the major concern in design is no longer the basin geometry, but the basin sub-surface geometry. In other word, we have to make sure that the infiltrating water rate can be sustained by the underground hydraulic gradient and conductivity.

**Example 2.** At the project site of Example 1, the soil porosity is 0.35 and has initial water content of 0.15. The distance to the local groundwater table is 10 feet. Design the basin geometry for storage volume of 0.219 acre-ft.

**Solution:** Under the saturated condition, the water storage volume in the 10-ft soil medium is:

$$d = 10 \times (0.35 - 0.15) = 2.0 \text{ feet of water}$$

Assuming that the basin is designed to have the brim-full depth of 2 feet, the basin area is determined as:

$$A_o = \frac{0.219}{2.0} = 0.11 \text{ acre-ft}$$

The final infiltration rate is 1.8 inch/hr in Example 1. Therefore, the drain time is:

$$T_d = \frac{2.0 \times 12}{1.8} = 13.3 \text{ hours}$$

So, the area of the basin is 0.11 acre. According to Example 1, the tributary area to this basin is 2.1 acres. The area of this basin is 4.7% of its tributary area.

## SUSTAINABILITY OF LONG-TERM OPERATION

After the soil medium is saturated, the infiltrating water directly recharges the groundwater table. If the seepage flow through the soils is slower than the infiltration rate on the land surface, the excess inflow will cause water mounting effect that may back up the system to cause a failure. The flow pattern below a circular basin is described in Figure 2. Such a three dimensional axially symmetric flow can be described by the stream function as:

$$\psi = \pi \frac{f_a}{D} r^2 y \quad (17)$$

in which  $r$  = radius at distance  $y$ . The streamlines below the basin are distributed as concentric circles with  $\Psi=0$  along the  $y$ -axis and  $\Psi$  = infiltration volume rate at the circumference of the basin bottom, i.e. Point  $C(r,y) = (R_o, D)$  in Figure 2. The infiltration volume rate released from the circular basin is:

$$Q = f_a \pi R_o^2 \quad (18)$$

in which  $Q$  = infiltration volume rate,  $f$  = soil infiltration rate, and  $R_o$  = radius of circular basin. For this flow field, the stream function is:

$$Q = \Psi \quad (19)$$

For a specified stream function between zero and  $Q$ , its streamline can be plotted through points  $(r, y)$  defined by Eq 19 on a vertical plane. To maintain the continuity of the flow, the radius of a horizontal cross section at specified  $y$ , its radius,  $r$ , is obtained by equating Eq 18 to Eq 19 to yield:

$$r = \sqrt{\frac{D}{y}} R_0 \quad (20)$$

The velocity components in the flow field can be depicted by the derivatives of Eq 19 with respect to  $r$  and  $y$  as:

$$u = \frac{1}{2\pi r} \frac{\partial \psi}{\partial y} = \frac{f_a}{2} \frac{r}{D} \quad (22)$$

and

$$v = \frac{1}{2\pi r} \frac{\partial \psi}{\partial r} = -f_a \frac{y}{D} \quad (23)$$

According to Darcy's law, the seepage rate across Section RP is:

$$Q = K_y i (\pi R^2) \quad (24)$$

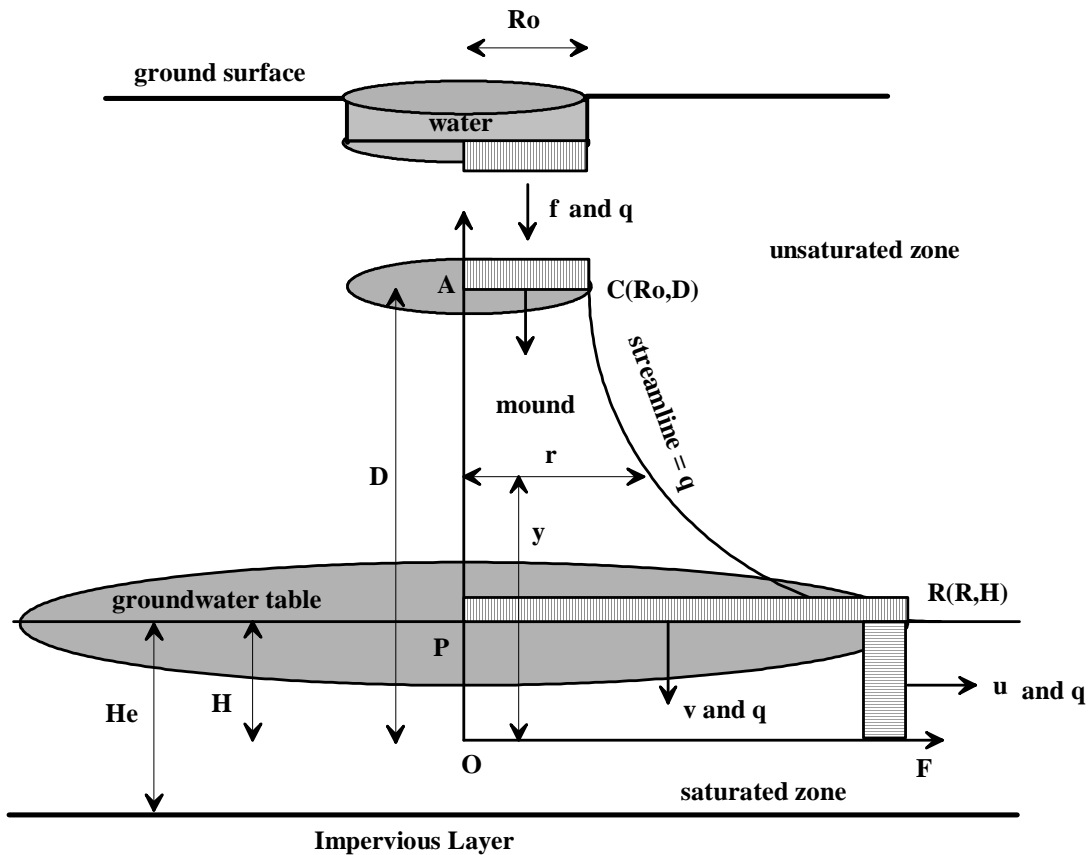


Figure 2 Flow Pattern under Circular Basin

At Section RP in Figure 2, the radius of the recharge area,  $R$ , at  $y = H$  is

$$R = \sqrt{\frac{D}{H}} R_o \quad (21)$$

in which  $i$  = vertical hydraulic gradient, and  $K_y$  = coefficient of vertical permeability. The downward velocity,  $v$ , at Section RP is:

$$v = \frac{Q}{\pi R^2} = K_y i \quad (25)$$

The downward hydraulic gradient in Eq 15.38 is  $i = -1$ . Since Eq 25 must be equal to Eq 23 at  $y = H$ , it yields

$$D = \lambda_y H \quad (26)$$

and

$$\lambda_y = \frac{f}{K_y} \quad (27)$$

At the outflow section, Section PF, the seepage rate is dictated by the hydraulic gradient between Point C and Section RF. Therefore, we have

$$Q = K_r (2\pi r y) \left(-\frac{\partial y}{\partial r}\right) \quad (28)$$

in which  $K_r$  = coefficient of radial permeability. Integrated Eq 28 from  $y = D$  to  $y = H$  and from  $r = R_o$  to  $r = R$  yields:

$$Q = \frac{K_r \pi}{\ln\left(\frac{R}{R_o}\right)} (D^2 - H^2) \quad (29)$$

in which  $H$  = active thickness of aquifer to be affected by recharge. Aided by Eq's 21, 26 and 29, the cross sectional average velocity,  $U$ , at Section PF is

$$u = \frac{Q}{2\pi R H} = K_r \frac{H}{R} \frac{(\lambda_y^2 - 1)}{\ln \lambda_y} \quad (30)$$

Similarly, Eq 30 must be consistent with Eq 22 at  $r = R_o$ . Aided by Eq's 21 and 26, the ratio of  $H/R_o$  can be derived by setting Eq 30 equal to Eq 22 as:

$$\frac{H}{R_o} = \sqrt{\frac{\lambda_r \ln \lambda_y}{2(\lambda_y^2 - 1)}} \quad (31)$$

and

$$\lambda_r = \frac{f}{K_r} \quad (32)$$

in which  $K_r$  = hydraulic conductivity in the radial direction. Substituting Eq 31 into Eq 21 yields

$$\frac{D}{R_o} = \lambda_y \sqrt{\frac{\lambda_r \ln \lambda_y}{2(\lambda_y^2 - 1)}} \quad (33)$$

Aided by Eq's 32, and 33, the required saturated depth is calculated as:

$$\frac{Y_o}{R_o} = \frac{D - H}{Y_o} \quad (34)$$

As aforementioned before, Eq 34 defines the minimum required vertical distance from the basin bottom to the groundwater table. If the site satisfies Eq 34, the infiltrating water will directly recharge the groundwater table; otherwise the water mounding effect will reduce the basin infiltrating efficiency.

**Example 3** Application of this model is illustrated by the design of a circular infiltration basin with a diameter of 68.0 ft. The distance to the groundwater table at the site is 16.5 feet. The basin will have a layer of loamy sand lining that has an infiltration rate of 1.80 inch/hr. From pumping well tests, the coefficient of soil permeability for the seepage flow is found to be 0.75 inch/hr. Evaluate the sustainability of this basin operation and propose a design infiltration rate for the site.

$$R_o = 34 \text{ ft and } f = 1.80 \text{ in/hr}$$

$$K_r = K_y = K = 0.75 \text{ in/hr}$$

$$Q = f\pi R_o^2 = \frac{1.80}{12.0 \times 3600.0} \times 3.1416 \times 34.0^2 = 0.15 \text{ cfs}$$

$$\lambda = \frac{1.80}{0.75} = 2.32$$

$$\frac{H}{R_o} = \sqrt{\frac{2.32 \times \ln(2.32)}{2(2.32^2 - 1)}} = 0.471 \text{ or } H = 16.1 \text{ ft}$$

$$\frac{D}{R_o} = 2.32 \sqrt{\frac{2.32 \times \ln(2.32)}{2(2.32^2 - 1)}} = 1.09 \text{ or } D = 37.2 \text{ ft}$$

$$Y_o = 37.2 - 16.1 = 21.1 \text{ feet}$$

The required saturated distance is greater than the available (21.1 > 16.5 ft). Therefore, the design infiltration rate must be reduced. Table 1 presents the cases with different f/K ratios. When f/K = 2.0, the required saturated depth is 16.34 feet. Therefore, it is suggested that the basin lining materials be designed to have an infiltration rate of 1.50 inch/hr.

f/K	f	H/R <sub>o</sub>	D/R <sub>o</sub>	Y <sub>o</sub> /R <sub>o</sub>	H	D	Y <sub>o</sub>
	in/hr				ft	ft	ft
1.20	0.90	0.50	0.60	0.10	16.95	20.34	3.39
1.50	1.13	0.49	0.74	0.25	16.77	25.15	8.38
<b>2.00</b>	<b>1.50</b>	<b>0.48</b>	<b>0.96</b>	<b>0.48</b>	<b>16.34</b>	<b>32.69</b>	<b>16.34</b>
2.50	1.88	0.47	1.17	0.70	15.88	39.7	23.82
3.00	2.25	0.45	1.36	0.91	15.34	46.29	30.86

Note: R<sub>o</sub> = 34 ft, K = 0.75 inch/hr

**Table 1 Design Infiltration Rates for Various f/K Ratios.**

## CLOSING

In summary, the storage volume of an on-site storm water detention system is solely determined by the tributary hydrology and the soil infiltration at the site. The basin geometry is then dictated by the soil porosity or the soil water storage volume. Before the soils become saturated, the operation of the basin relies on the soil water storage capacity. Under the saturated condition, the operation of the basin relied on the soil conductivity that depends on the subsurface hydraulic gradient and soil permeability. This paper identified the serious negligence in the design of storm water on-site disposal facilities, and then derived the on-site constraints based on the surface and subsurface geometries for the designs of porous pavements, riprap trenches, infiltration beds, retention pools, detention systems, wetlands, and recharging basins. The complicated flow systems can be simplified by the potential flow approach. The potential flow model estimates the two-dimensional flow field beneath the basin. The model does not require detailed site specifics. It is simple for use and provides reasonable assessments on infiltrating water movement and mounding impacts. It is a useful tool at the planning stage when little design information is available. The potential flow model ignores the friction due to the nature of laminar flow in groundwater movement. For long term forecasting of storm water retention process, it is necessary to further calibrate the model by field observations.

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